SEATTLE REGIONAL MEETING—NOVEMBER 5-6, 1957

JULY 1957
No. 1
VOL. 29

JOURNAL
OF THE
AMERICAN CONCRETE INSTITUTE

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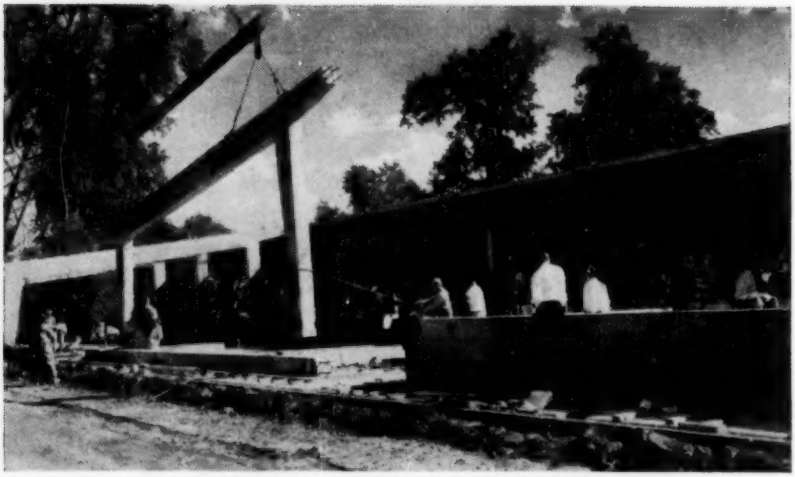

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PROCEEDINGS VOL. 54

JOURNAL
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Bound volumes 1 to 52 of PROCEEDINGS of the AMERICAN CONCRETE INSTITUTE (1905 to 1955) are for sale as far as available at prices to be had on inquiry of the Secretary-Treasurer. Special prices apply for members ordering bound volumes in addition to the monthly Journal.

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the ACI JOURNAL

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Coming next month in the JOURNAL

LEAD ARTICLE in the August JOURNAL will be "Proposed Test Procedure to Determine Relative Bond Value of Reinforcing Bars," reported by ACI Committee 208, whose chairman is C. P. Siess.

PAUL RICE, ACI's technical director, has prepared "Design of Concrete Floors on Ground for Warehouse Loadings," also slated for August publication.

LIGHTWEIGHT CONCRETE was used in the "Multispan Prestressed Folded Plate Roof for Langendorf Bakery," to be described by John J. Driskell.

RESEARCH STUDY of the "Effect of Tobermorite on the Mechanical Strength of Autoclaved Portland Cement-Silica Mixtures," will be presented by L. Dean Sanders and W. J. Smothers.

TESTS CONDUCTED at the University of Colorado will be described by Gene M. Nordby and William J. Venuti in "Fatigue and Static Tests of Steel Strand Prestressed Beams of Expanded Shale Concrete and Conventional Concrete."

●

Discussion

Beginning with the July issue, No. 1 of the new *Proceedings* volume, discussion of ACI papers and reports will be published quarterly, in March and June JOURNALS, and in Part 2 of September and December JOURNALS. A uniform 3-month period will be allowed for submitting discussion, with closure dates indicated on the first and last page of every paper.

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Title No. 54-1

Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-57)*

Reported by ACI Committee 214

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SYNOPSIS

Statistical methods provide valuable tools for assessing results of strength tests, and such information is also of value in refining design criteria and specifications. The report discusses briefly the numerous variations that occur in the strength of concrete and presents statistical methods which are useful in interpreting these variations. Criteria are offered that can be used to establish specifications and maintain required uniformity.

INTRODUCTION

The primary function of compression tests of field concrete is to insure production of uniform concrete of desired strength and quality. Concrete being a hardened mass of heterogeneous materials is subject to the influence of numerous variables. Characteristics of each of the ingredients of concrete may cause variations depending upon their uniformity. Variations may also be introduced by practices used in proportioning, mixing, transporting, placing, and curing. In addition to the variations which exist in concrete itself, strength variations will also be introduced in fabrication, testing, and care of test specimens. Variations in the strength of concrete must be accepted; but consistent concrete of adequate quality can be produced with confidence if proper control is maintained, test results are properly interpreted, and limitations are considered.

*Adopted as a standard of the American Concrete Institute at its 53rd annual convention, Feb. 26, 1957, as reported by Committee 214; ratified by letter ballot June 1, 1957. Title No. 54-1 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 1, July 1957, *Proceedings* V. 54. Separate prints in covers are available at 60 cents each to ACI members (75 cents to nonmembers). **Discussion** (copies in **triplicate**) should reach the Institute not later than Oct. 1, 1957. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

Proper control is achieved by the use of satisfactory materials, properly mixing these materials into economical concrete of desired quality, and good practices in transporting, placing, curing, and protecting the fresh concrete. Although the complex variables of concrete preclude complete uniformity, excessive variation of concrete strength signifies inadequate concrete control. Improvement in control may permit a reduction in the cost of concrete since the average strength can be adjusted more closely to specification requirements.

Compressive strength is not necessarily the most critical factor in proportioning concrete mixes since other factors, such as durability, may impose lower water-cement ratios than are required to meet strength requirements. In such cases strength will of necessity be in excess of structural demands. Strength tests are valuable in such circumstances, however, since with established mix proportions, variations in strength are indicative of variations in other properties.

Test specimens indicate potential rather than actual strength of a structure and poor workmanship in placing and curing may cause strength reductions which are not reflected in tests. Wherever practicable, conclusions on strength of concrete should be derived from a pattern of tests from which the characteristics and uniformity of the concrete can be more accurately estimated. To place too much reliance on too few tests may result in erroneous conclusions.

Statistical methods have not been used to any great extent in the design and construction of concrete structures, except by large construction agencies. These methods provide tools of considerable value in assessing results of strength tests and such information is also of value in refining design criteria and specifications. This report discusses briefly variations that occur in the strength of concrete, presents statistical methods which are useful in interpretation of these variations, and offers criteria that can be used to establish specifications and maintain required uniformity. Although this report is written for compression tests, the statistical methods presented may also be applied to flexural strength and other tests of concrete.

VARIATIONS IN STRENGTH

The magnitude of variations in the strength of concrete test specimens depends upon how well the materials, concrete manufacture, and tests are controlled. Differences in strength can be traced to two fundamentally different sources as shown in Table 1: (a) difference in strength-producing properties of the concrete mixture, and (b) apparent difference in strength caused by discrepancies in tests.

Properties of concrete

It is well established that strength of concrete is governed to a large extent by the water-cement ratio. The first criterion for producing concrete of uniform strength, therefore, is a uniform water-cement ratio. Since the

TABLE 1—PRINCIPAL SOURCES OF STRENGTH VARIATION

Variations in the properties of concrete	Discrepancies in testing methods
Changes in water-cement ratio: Poor control of water content Excessive variation of moisture in aggregate	Inconsistent sampling procedures
Variations in water requirement: Aggregate grading Nonuniform materials	Nonuniform fabrication techniques: Amount of compaction Excessive handling of sample Care of fresh cylinders
Variations in characteristics and proportions of ingredients: Aggregates Cement Pozzolans Admixtures	Change in curing: Temperature variation Variable moisture
Variations in transporting, placing, and compaction	Poor testing procedures: Cylinder capping Compression tests
Variations in temperature and curing	

quantity of cement can be measured accurately, the problem of maintaining a uniform water-cement ratio is primarily a problem of controlling the water content. This problem is aggravated because of the variable quantity of free moisture in aggregates.

The uniformity of concrete can be no better than the uniformity of concrete aggregates, cement, and admixtures used since each will contribute to the variations of concrete strength. The temperature of fresh concrete influences its water-cement ratio for a given consistency and consequently contributes to strength variation. Construction practices may cause variations in strength due to inadequate mixing, poor compaction, delays, and improper curing.

Use of admixtures presents additional problems in maintaining uniformity of strength since each admixture adds another variable to concrete. Accelerators, retarders, pozzolans, and air-entraining agents must be carefully controlled and allowance made for their influence on strength.

Testing methods

Concrete tests may or may not include all the variations in strength of concrete in place depending on what variables are introduced after test specimens have been made. On the other hand, discrepancies in sampling, fabrication, curing, and testing of specimens may indicate variations in strength which do not exist in the concrete in place. The project is unnecessarily penalized when variations from this source are excessive. Good testing methods will reduce these variations and standard testing procedures such as those described in ASTM standards should be established and followed without variation.

The importance of using accurate compression testing machines and other laboratory equipment should need no emphasis since test results can be no more accurate than the equipment used. *Uniform test results are not necessarily accurate test results.* Laboratory equipment and procedures should be calibrated and checked periodically.

ANALYSIS OF STRENGTH DATA

To obtain maximum information, a sufficient number of field compression tests should be made to be representative of the concrete produced and appropriate statistical methods should be used to interpret the test results. Statistical methods provide the best basis for assessing from such results the potential quality and strength of the concrete in a structure and expressing results in the most useful form.

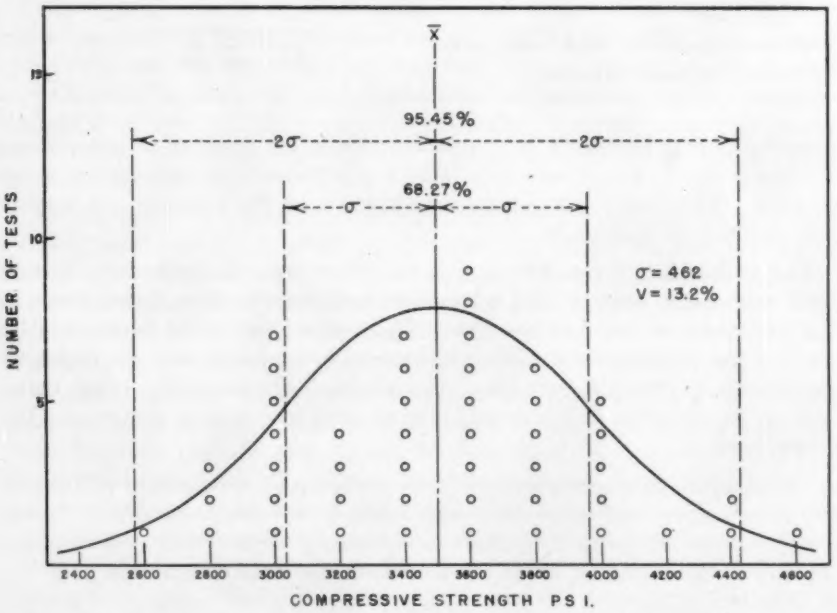

Fig. 1—Normal frequency distribution of strength data from 46 tests

Statistical functions

The strength of concrete test specimens on controlled projects can be assumed to fall into some pattern of the normal frequency distribution curve as illustrated in Fig. 1. For more complete statistical studies, determination of the actual shape of the curve may be desirable. Where there is good control, the strength values will be bunched close to the average, and the curve will be tall and narrow. As the variations in strength increase, the values spread and the curve becomes low and elongated (Fig. 2). Certain functions of the normal frequency curve are useful in evaluating strength data:

Average, $\overline{X}$ — The average strength of all cylinders.

$$\overline{X} = \frac{X_1 + X_2 + X_3 + \ldots + X_n}{n} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

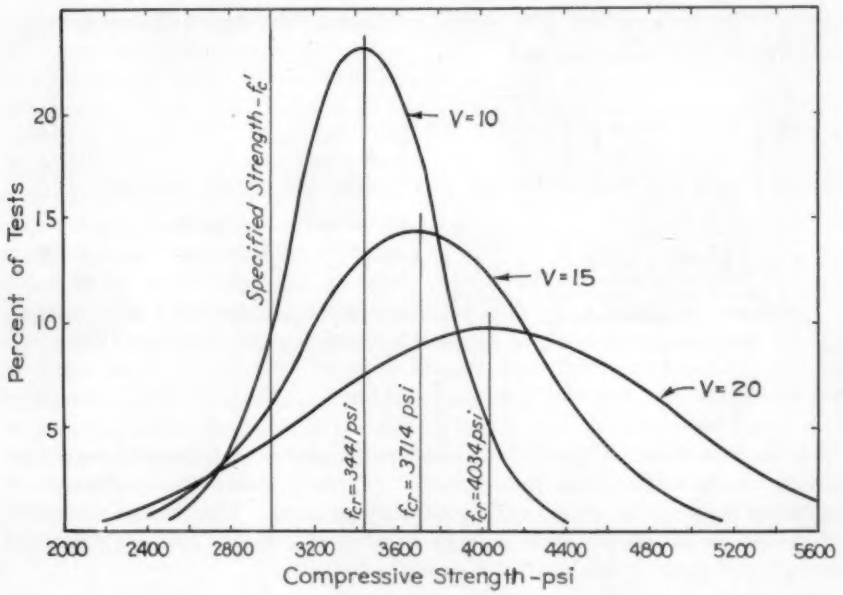

Fig. 2—Normal frequency curves for coefficients of variation of 10, 15, and 20 percent

Required average strength f_{cr} based on a probability of one in ten that a test will fall below a specified strength f_c' of 3000 psi

Where $X_1, X_2, X_3 \ldots X_n$ are the strength results of individual specimens and n is the total number of specimens tested.

Standard deviation, σ—The most generally recognized measure of dispersion is the root-mean-square deviation of the strengths from their average. Referring to Fig. 1 the standard deviation is the radius of gyration of the area under the theoretical probability curve about the center. The standard deviation is found by extracting the square root of the average of the squares of deviations of individual strengths from their average.*

$$\sigma = \sqrt{\frac{(X_1 - \overline{X})^2 + (X_2 - \overline{X})^2 + \ldots + (X_n - \overline{X})^2}{n}} \quad \ldots\ldots\ldots\ldots (2)$$

For ease of computation, when a calculating machine is available, standard deviation can be obtained by dividing the sum of the squares of individual

*Some texts on statistics refer to σ as the standard deviation of the universe (infinite number of tests) and s as the standard deviation of a sample. In computing s instead of σ in Eq. (2), n is replaced by $(n - 1)$. This practice was not followed in the manual of ASTM Committee E-11 on Quality Control of Materials (*ASTM Special Technical Publication* No. 15-C) and is significant only when the number of samples is small (less than 30). Generally, control of concrete uniformity involves a large number of samples collected over a period of time and the difference between methods is negligible. On the other hand, the number of samples is usually small (two or three tests) in computing the within-test standard deviation σ_1. Eq. (5) is useful in this case since the factor $1/d_2$ and a reliable value of $\overline{R}$ produce a true or unbiased estimate of σ_1 as distinct from observed values as in Eq. (2), (3), and (7). The values of Table 2 were established from a large number of tests on the basis of the formulas as presented.

strengths by the numbers of cylinders, subtracting the square of their average, and extracting the square root.

$$\sigma = \sqrt{\frac{X_1^2 + X_2^2 + X_3^2 + \ldots + X_n^2}{n} - \bar{X}^2} \quad \ldots\ldots (3)$$

or rearranged to bring the fraction from under the square root sign:

$$\sigma = \frac{1}{n}\sqrt{n(X_1^2 + X_2^2 + X_3^2 + \ldots + X_n^2) - (X_1 + X_2 + X_3 + \ldots + X_n)^2} \quad (3a)$$

Coefficient of variation, V—The standard deviation expressed as a percentage of the average strength is called the coefficient of variation.

$$V = \frac{\sigma}{\bar{X}} \times 100 \quad \ldots\ldots (4)$$

This function makes it possible to express the degree of dispersion on a percentage basis rather than the absolute. Table 2 shows the coefficients of variation that can be expected on controlled projects. The ratings of control are based on experience from a large number of projects and are presented as a general guide in evaluation of concrete control.

TABLE 2—STANDARDS OF CONCRETE CONTROL

Class of operation	Coefficients of variation for different control standards			
	Excellent	Good	Fair	Poor
Over-all variations:				
General construction	Below 10.0	10.0 to 15.0	15.0 to 20.0	Above 20.0
Laboratory control	Below 5.0	5.0 to 7.0	7.0 to 10.0	Above 10.0
Within-test variations:				
Field control	Below 4.0	4.0 to 5.0	5.0 to 6.0	Above 6.0
Laboratory control	Below 3.0	3.0 to 4.0	4.0 to 5.0	Above 5.0

Note: These standards represent the average for 28-day cylinders computed from a large number of tests Different values for other than average concretes can be expected.

Range, R—Range is found by subtracting the lowest strength in any group of tests from the highest strength. Range is useful in computing the within-test standard deviation discussed in the following section.

Batch variations

As mentioned previously, variations in results of strength tests can be traced to two different sources: (a) properties of the concrete mixture, and (b) discrepancies in testing methods. It is possible by analysis of variance to compute the variations attributable to each source.

Within-test variation—The variation in strength of concrete within a single batch can be found by computing the variations of a group of cylinders fabricated from that batch. It is also convenient to assume that a test

sample of concrete is uniform and any variation between companion cylinders fabricated from a given sample is caused by fabricating, curing, and testing discrepancies. Samples taken from different parts of a batch will include variations due to mixer inefficiency. Companion specimens fabricated from samples from different locations in the mixer can be used therefore to differentiate between mixer efficiency and testing efficiency.

TABLE 3—FACTORS FOR COMPUTING WITHIN-TEST STANDARD DEVIATION*

Number of specimens	d_2	$1/d_2$
2	1.128	0.8865
3	1.693	0.5907
4	2.059	0.4857
5	2.326	0.4299
6	2.534	0.3946
7	2.704	0.3698
8	2.847	0.3512
9	2.970	0.3367
10	3.078	0.3249

*From Table B2, "Manual on Quality Control of Materials," *ASTM Special Technical Publication* No. 15-C.

A single batch of concrete provides meager data for statistical analysis and companion cylinders from at least ten samples of concrete are recommended to establish reliable values for $\overline{R}$. The within-test standard deviation and coefficient of variation can be conveniently computed as follows:

$$\sigma_1 = \frac{1}{d_2}\overline{R} \quad \text{.....(5)}$$

$$V_1 = \frac{\sigma_1}{\overline{X}} \times 100 \quad \text{.....(6)}$$

where

σ_1 = within-test standard deviation

$\frac{1}{d_2}$ = a constant depending upon the number of cylinders in each group (Table 3)

$\overline{R}$ = average range of groups of companion cylinders

V_1 = within-test coefficient of variation

$\overline{X}$ = average strength

Cumulative distribution

If the theory of the "weakest link" is applied to concrete, the number of tests lower than the desired strength is more important in computing the load-carrying capacity of concrete structures than the average strength obtained. It is impractical, however, to specify minimum strength since the law of normal probability indicates we can expect one strength out of every six tests to be lower than the standard deviation (σ) below average ($\overline{X}$), one out of every 44 will be lower than 2σ below average, and one out of 741 will be lower than 3σ below average.

The cumulative distribution curve is plotted by cumulating the number of tests below any given strength. Fig. 3 shows a group of cumulative distribution curves based on percentage and plotted as straight lines on the probability scale. This chart provides a convenient means of securing cumulative probability data when the coefficients of variation are known.

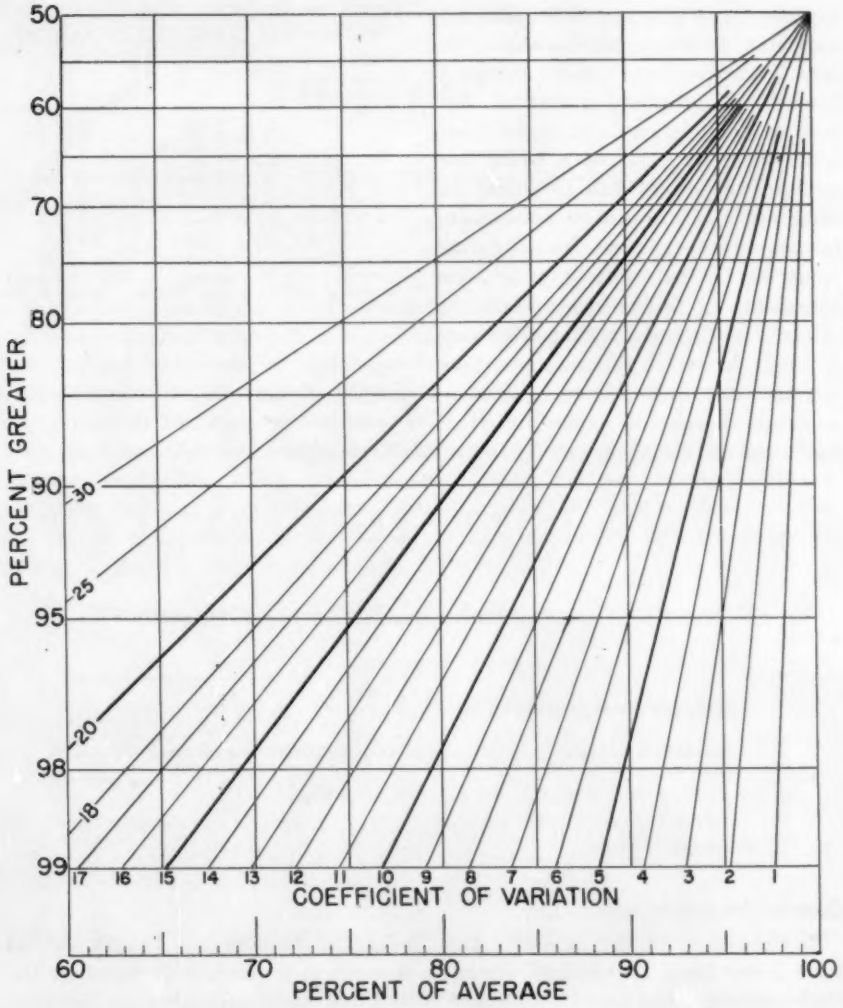

Fig. 3—Cumulative distribution curves for different coefficients of variation

Quality-control charts

Quality-control charts have been used by manufacturing industries for many years as an aid to uniformity and efficiency in production. Methods are well established for the setting up of such charts and are outlined in convenient form in the ASTM "Manual on Quality Control of Materials." Based on the pattern of previous results and limits established therefrom, trends become apparent as soon as new results are plotted. Points which fall outside the calculated limits are indicative that something has affected the

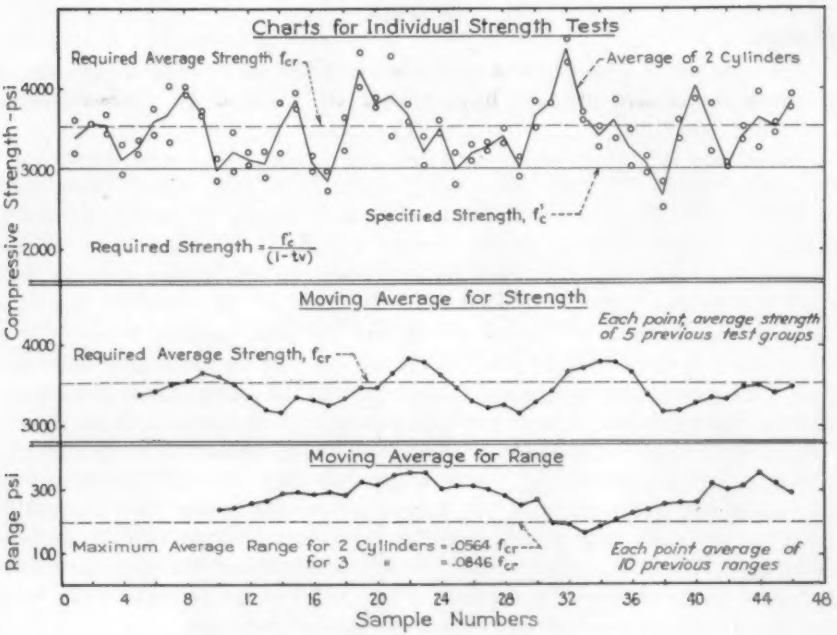
Charts for Individual Strength Tests
Required Average Strength f_{cr}
Average of 2 Cylinders
Specified Strength, f'_c
Required Strength = $\frac{f'_c}{(1-tv)}$
Compressive Strength - psi
4000
3000
2000
Moving Average for Strength
Each point average strength of 5 previous test groups
Required Average Strength, f_{cr}
4000
3000
Moving Average for Range
Each point average of 10 previous ranges
Maximum Average Range for 2 Cylinders = .0564 f_{cr}
for 3 " = .0846 f_{cr}
Range psi
300
100
0 4 8 12 16 20 24 28 32 36 40 44 48
Sample Numbers

Fig. 4—Quality-control charts for concrete

control of the process. Such charts are recommended wherever concrete is in continuous production over considerable periods.

Three simplified charts prepared specifically for concrete control are illustrated in Fig. 4. (As pointed out in Example 4, p. 18, this is not particularly good control.) While these do not contain all the features of formal control charts they should prove useful to the engineer, architect, and plant superintendent.

(a) A chart in which the results of all strength tests are plotted as received. The line for the required average strength is established as indicated in the section on criteria, p. 10, and the specified design strength shows the number of low tests.

(b) Moving average for compressive strength where the average is plotted for the previous five sets of two companion cylinders for each day or shift and the specified strength in this case is the lower limit. This chart is valuable in indicating trends and will show the influence of seasonal changes, changes in materials, etc. The number of tests averaged to plot moving averages with an appropriate lower limit (Table 5) can be varied to suit each job.

(c) Moving average for range where the average range of the previous ten groups of companion cylinders is plotted each day or shift. The maximum average range allowable for good laboratory control is also plotted. Maximum average range is determined as discussed in the section on criteria.

Fig. 4 shows charts a, b, and c for 46 tests. To be fully effective charts should be maintained throughout the entire job.

CRITERIA

Strength

The strength of concrete test specimens and the load carrying capacity of concrete structures have been correlated by tests made with full-scale structural members.

Indications of structural strength are also obtained by testing prototypes, by coring, and by dynamic and other nondestructive type tests. These tests are not widely used, however, and the strength of control cylinders is generally the only tangible evidence of the quality of concrete in a structure. Compressive strength data are therefore important in establishing criteria, but their limitations must be considered. Because of the possible disparity between the strength of test specimens and the load carrying capacity of a structure it is dangerous to place too much reliance on inadequate strength data. It is also wrong to conclude that the strength of a structure is in jeopardy when a single test fails to meet specified strength requirements. As previously indicated, random variations and occasional failures to comply with strength requirements are inevitable. Accordingly inflexible strength requirements are unrealistic and control of the pattern of results rather than individual values is the most appropriate basis for both specifications and the general assessment of results. This is the reason why statistical concepts have such potential value in concrete control. From these concepts there have been developed tools to measure and indices to control variation.

The consequences of a localized zone of low-strength concrete in a structure depend on many factors; included are the probability of early overload, the location and magnitude of the low-quality zone in the structural unit, the degree of reliance placed on compressive strength in design, the initial cause of the low strength, and the consequences, economically and otherwise, of structural failure. As a general guide it is the opinion of the committee that a reasonable control of structural concrete would be provided if the probability of any test falling below f_c' used in design is no greater than 0.10 (1 in 10). This tolerance of test failure does not imply acceptance of consecutive failures in 10 percent of the structure, but must be expressed as a continuous control rather than an over-all percentage. Greater probability of low-strength specimens is allowable in other classes of concrete (Table 5) but the final criterion adopted is obviously a matter for the designer's decision based on his intimate knowledge of the conditions that are likely to prevail.

To satisfy strength performance requirements expressed in this fashion the average strength of concrete must obviously be in excess of f_c', the degree of excess strength depending on the expected uniformity of concrete production and the allowable proportion of low tests. The required average strength, f_{cr}, for any design can be approximated directly from Fig. 5 or computed as follows:

$$f_{cr} = \frac{f_c'}{(1 - tV)} \qquad (7)^*$$

*The required average modulus of rupture for beams can be obtained using Eq. (7) by substituting the specified modulus of rupture (SMR) for f_c' and using an appropriate coefficient of variation for flexure tests.

where

f_{cr} = required average strength
f_c' = design strength specified
t = a constant depending upon the proportion of tests that may fall below f_c' and the number of samples used to establish V (Table 4)
V = forecasted value of the coefficient of variation expressed as a fraction

Table 4 indicates a value for t of 1.282 for a probability of 1 in 10 of a test being low, and Table 2 indicates that for good field control the coefficient of variation should not be greater than 15 percent. In this case f_{cr} is equal to 1.24 f_c' and for a structural design strength of 3000 psi the mixture will be proportioned for an average strength of 3720 psi. This value of f_{cr} could be reduced with better control and must be increased for poorer control as illustrated in Fig. 2. Where a batch of low-strength concrete is not so critica

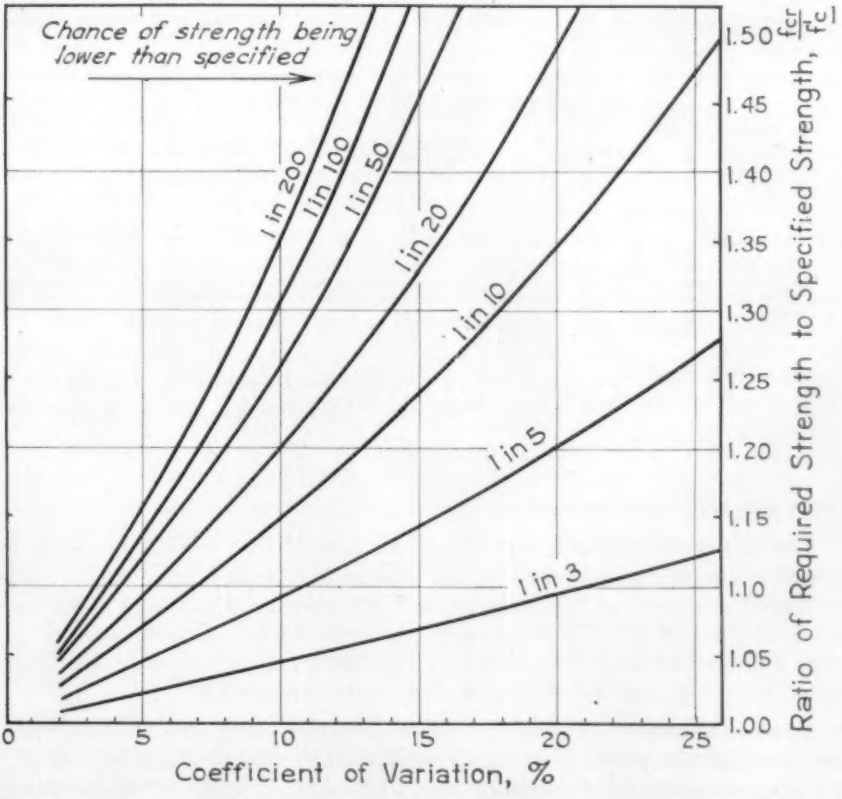

Fig. 5—Ratio of required average strength f_{cr} to specified strength f_c' for various coefficients of variation and chances of falling below specified strength

This chart is based on the distribution of the normal variate while Eq. (7) is based on the distribution of "Student's t." For a small number of tests Eq. (7) requires a higher, hence a more reliable value for f_{cr}

TABLE 4—VALUES OF t*

No. of samples minus 1†	Percentage of tests falling within the limits $\bar{X} \pm t\sigma$							
	50	60	70	80	90	95	98	99
	Chances of falling below lower limit							
	2.5 in 10	2 in 10	1.5 in 10	1 in 10	1 in 20	1 in 40	1 in 100	1 in 200
1	1.000	1.376	1.963	3.078	6.314	12.706	31.821	63.657
2	0.816	1.061	1.386	1.886	2.920	4.303	6.965	9.925
3	0.765	0.978	1.250	1.638	2.353	3.182	4.541	5.841
4	0.741	0.941	1.190	1.533	2.132	2.776	3.747	4.604
5	0.727	0.920	1.156	1.476	2.015	2.571	3.365	4.032
6	0.718	0.906	1.134	1.440	1.943	2.447	3.143	3.707
7	0.711	0.896	1.119	1.415	1.895	2.365	2.998	3.499
8	0.706	0.889	1.108	1.397	1.860	2.306	2.896	3.355
9	0.703	0.883	1.100	1.383	1.833	2.262	2.821	3.250
10	0.700	0.879	1.093	1.372	1.812	2.228	2.764	3.169
15	0.691	0.866	1.074	1.341	1.753	2.131	2.602	2.947
20	0.687	0.860	1.064	1.325	1.725	2.086	2.528	2.845
25	0.684	0.856	1.058	1.316	1.708	2.060	2.485	2.787
30	0.683	0.854	1.055	1.310	1.697	2.042	2.457	2.750
∞	0.674	0.842	1.036	1.282	1.645	1.960	2.326	2.576

*Values of t extracted from table originally produced by Fisher and Yates, "Statistical Tables for Biological Agriculture and Medical Research."
†Degrees of freedom.

Other values of t for $n - 1 = \infty$

Percentage within $\bar{X} \pm t\sigma$	Chances of falling below lower limit	t
40	3 in 10	0.524
68.27	1 in 6.3	1.000
95.45	1 in 44	2.000
99.73	1 in 741	3.000

Values of t increase for small samples due to the unreliability of small samples to establish a true estimate of σ. The advantage of establishing V in Eq. (7) from a large number of tests becomes apparent in the reduction of t and f_{cr}.

and two low test cylinders in ten can be tolerated, t equals 0.842 and $f_{cr} = 1.14\, f_c'$ or 3430 psi for a design strength of 3000 psi and a coefficient of variation of 15 percent.

Tests and specimens required

For any particular job, a sufficient number of tests should be made to insure accurate representation of the concrete. Concrete tests can be made either on the basis of time elapsed or cubic yardage placed and conditions on each job will determine the most practical number of tests needed. A test is defined as the average strength of all specimens of the same age fabricated from a sample taken from a single batch of concrete.

Centralized project—A project where all concrete operations are supervised by one engineer provides an excellent opportunity for uniform control and for accurate estimates of reliability with a minimum of tests. After operations are progressing smoothly, tests taken each day or shift, depending upon the volume of concrete produced, are sufficient to obtain data which are representative of all structures. Under adequate control of all operations, it is unnecessary to test the concrete at the point of placement unless dictated

by unusual problems. In general, it is advisable to make a sufficient number of tests so that each different type of concrete placed during any one day will be represented by two standard 6 x 12-in. cylinders to be tested at 28 days. Single specimens taken from two different batches each day will provide more reliable information on over-all variations, but it is usually desirable to make companion specimens from the same sample to obtain a check on the within-test variation.

Producer—The number of control tests required to properly evaluate the operations of a concrete mixing plant is similar to that of a centralized project since operations and procedures are uniform even though concrete may be supplied to many different structures. Problems encountered and control desired will dictate the number of tests for a given mixing plant but if operations are progressing uniformly, reliable information on over-all plant operation over a period can be obtained with one 28-day cylinder each day or each shift for each different type of concrete. If information regarding the accuracy of tests is desired, companion specimens will be required to differentiate the variations due to testing from plant variations.

Consumer—The consumer or owner, usually represented by an architect or engineer, has more than academic interest in the strength of concrete since the safety and load carrying capacity of the structure may be impaired by poor control and batches of low-strength concrete. If all operations are controlled by one engineer, the problem is simplified as indicated for a centralized project. The popular practice of purchasing concrete from a separate producer and contracting the testing work separately complicates concrete testing and control.

The consumer is not only interested in the concrete as it leaves the plant but is interested in every operation until the concrete is placed in the structure and adequately cured. In addition to mixing-plant-control tests, the engineer (architect) should require performance tests to insure concrete of desired properties. Performance tests should be taken at the point of placement and companion specimens should be made for greater accuracy and to establish responsibility for variation between the producer and the testing laboratory. The contractor may also contribute to variations with poor construction practices, hence correlation between control tests taken at the producer's plant and performance tests at the point of placement will help establish responsibility.

The number of specimens required by the engineer (architect) should be based on established standards but may be reduced as the reliability of the producer, the laboratory, and the contractor are established.

Testing laboratory—The testing laboratory has the responsibility of making accurate tests, and concrete will be penalized unnecessarily if tests show greater variations than actually exist. Since the range between companion specimens from the same sample can be assumed to be the responsibility of

the testing laboratory, a control chart for ranges (Fig. 4) should be maintained by the laboratory as a check on the uniformity of its operations. The range between companion cylinders depends on the number of specimens in the group and the within-test variation. This relationship is expressed by the following equation:

$$\bar{R}_m = f_{cr} V_1 d_2 \quad (8)$$

The within-test coefficient of variation V_1 should not be greater than 5 percent for good control (Table 2), and the estimate of the corresponding average range will be:

$\bar{R}_m = (0.05 \times 1.128) f_{cr} = 0.05640 f_{cr}$ for groups of two companion cylinders
$\bar{R}_m = (0.05 \times 1.693) f_{cr} = 0.08465 f_{cr}$ for groups of three companion cylinders
$\bar{R}_m$ becomes the maximum for average range in control chart (c) of Fig. 4.

Type of specimens required—A cylinder of representative concrete 6 in. in diameter and 12 in. high which has been moist cured for 28 days is generally recognized as a standard specimen for strength and uniformity of concrete if the coarse aggregate does not exceed 2 in. in nominal size. Many times, particularly in the early stages of a job, it becomes necessary to estimate the strength of concrete being produced before the 28-day strength results are available. Concrete cylinders from the same batch should be made and tested at 7 days or earlier ages if desired. The 28-day strength can be estimated by extrapolating early test data.

The strength of concretes at later ages, particularly where pozzolans or cements of slow strength gain are used, is more realistic than the standard 28-day strength. Some structures will not be loaded until concrete has been allowed to cure for longer periods and advantage can be taken of strength gain after 28 days. Some concretes have been found to produce less than 50 percent of their ultimate strength at 28 days. If design is based on strength at later ages, it becomes necessary to correlate these strengths with standard 28-day cylinders since it is not practicable to use later age specimens for concrete control. If possible the correlation should be established by laboratory tests before construction starts. If mixing plants are located in one place for long enough periods, it is advisable to establish this correlation for reference even though later age concrete is not immediately involved.

Curing concrete test specimens at the construction site and under job conditions is sometimes recommended since it is more representative of the curing given the structure; these special tests should not be confused with, nor replace, standard control tests. Tests of job-cured specimens may be highly desirable and are necessary when determining the time of form removal, particularly in cold weather, and when establishing the strength of steam-cured concrete pipe, block, and structural members.

The potential strength and uniformity of concrete can be established by standard 6 x 12-in. cylinders made and cured under standard conditions. Strength specimens of concrete made or cured under other than standard

conditions provide additional information but should be analyzed and reported separately.

Rejection of doubtful specimens—The practice of arbitrary rejection of test cylinders which appear "too far out of line" is not recommended since the normal pattern of probability establishes the possibility of such results. Discarding tests indiscriminately could seriously distort the strength distribution, making analysis of results less reliable.

It occasionally happens that one of a set of strength values deviates so far from the mean as to be highly improbable. The committee recommends that a specimen from a test of three or more specimens be discarded if its deviation from a test mean is greater than 3 σ and should be accepted with suspicion if its deviation is greater than 2 σ. If questionable variations have been observed during fabrication, curing, or testing of a specimen, it should be rejected. The test average should be computed from the remaining specimens.

A test (average of all specimens of a sample) should never be rejected unless the specimens are known to be faulty, since it represents the best available estimate for the sample. Rejection of low tests would nullify strength specifications limits shown in Table 5.

SPECIFICATIONS FOR CONCRETE STRENGTH

Based on the foregoing considerations, Table 5 is presented as a guide in establishing strength specifications. These suggested limits are based on the premise that concrete is proportioned to produce an average strength equal

TABLE 5—GUIDE FOR CONCRETE STRENGTH SPECIFICATIONS

Number of tests	Minimum strength requirements for the averages of consecutive tests					
	Structural concrete		Pavements		Dams and irrigation works	Miscellaneous concrete where strength is not critical
	Ultimate strength design	General	Compressive strength	Flexural strength		
1	0.75 f_c'	0.70 f_c'	0.65 f_c'	0.65 MR	0.65 f_c'	0.62 f_c'
2	0.92 f_c'	0.86 f_c'	0.80 f_c'	0.80 MR	0.80 f_c'	0.75 f_c'
3	1.00 f_c'	0.93 f_c'	0.86 f_c'	0.86 MR	0.86 f_c'	0.82 f_c'
4	1.04 f_c'	0.97 f_c'	0.90 f_c'	0.90 MR	0.90 f_c'	0.85 f_c'
5	1.07 f_c'	1.00 f_c'	0.92 f_c'	0.92 MR	0.92 f_c'	0.88 f_c'
6	1.09 f_c'	1.02 f_c'	0.94 f_c'	0.94 MR	0.94 f_c'	0.89 f_c'
Probability of any test falling below f_c'	1 in 20	1 in 10	2 in 10	2 in 10	2 in 10	3 in 10
Questionable number of consecutive tests below f_c'*	1	2	3	3	3	4
Number of consecutive tests which constitute failure	2	3	4	4	4	5

*The probability of obtaining the indicated questionable number of consecutive low tests ranges from 1 to 5 in 100. This does not constitute failure of these requirements but at the same time casts some doubt on the adequacy of strength or uniformity of control.

to f_{cr} and have approximately the same probability of failure in each case. Failure to meet these limits will be an indication that the average strength does not equal f_{cr}. This could be caused by lower strength or poorer control than expected, or both. In either case the average strength should be increased.

The requirements for ultimate strength design are more restrictive and require closer control since load factors are selected on the premise that the strength of concrete in a structure will not be less than the assumed value of f_c' and do not allow for the variability of concrete strength.

Most specifications for concrete strength require that a test be comprised of three specimens from the same sample of concrete. Three specimens are necessary to obtain a reliable average for a given sample but it should also be emphasized that more reliable strength data are obtained from three tests of one cylinder each than from one test of three cylinders.

EXAMPLES

The mechanics of analyzing concrete strength data can be explained in more detail by using hypothetical examples of typical field jobs.

Example 1

Table 6 lists concrete strength data taken at a commercial mixing plant over a period of 2 months. It is desired to determine the degree of uniformity being obtained in comparison with general practice and to establish average

TABLE 6—EVALUATION OF UNIFORMITY OF PRODUCTION

Test No.	28-day strength, psi		Range, psi	Test No.	28-day strength, psi		Range, psi
	Cylinder 1	Cylinder 2			Cylinder 1	Cylinder 2	
1	3600	3190	410	24	3410	3500	90
2	3560	3550	10	25	2780	3180	300
3	3420	3670	250	26	3090	3300	210
4	2920	3300	380	27	3210	3310	100
5	3330	3190	140	28	3490	3300	190
6	3750	3400	350	29	2890	3040	150
7	4030	3920	110	30	3810	3500	310
8	4010	3540	470	31	3830	3800	30
9	3690	3650	40	32	4590	4370	220
10	2830	3120	190	33	3600	3700	100
11	2950	3450	500	34	3510	3260	250
12	3040	3200	160	35	3350	3840	490
13	3210	2900	310	36	3020	3480	460
14	3810	3190	620	37	3150	2940	210
15	3950	3730	220	38	2510	2820	310
16	3150	2960	190	39	3370	3600	230
17	2930	2700	230	40	4210	3860	350
18	3610	3210	400	41	3800	3200	600
19	4440	4000	440	42	3000	3050	50
20	3870	3770	100	43	3530	3340	190
21	4390	3600	790	44	3950	3250	700
22	3560	3790	230	45	3590	3440	150
23	3040	3400	360	46	3910	3760	150

n = 92
ΣX = 317,960
$\Sigma X/n$ = 3456 psi
ΣX^2 = 1,114,248,800
$\Sigma X^2/n$ = 12,111,400
$\overline{X}^2$ = 11,943,936

σ^2 = 167,464
σ = 409
V = 11.8 percent
$\overline{R}$ = 277
σ_1 = 277 × 0.8865 = 246
V_1 = 7.1 percent

strengths required by this plant to meet various specifications. Companion specimens have also been taken to evaluate uniformity of testing.

The computations listed with Table 6 show that an average strength of 3456 psi was obtained during the 2-month period.

The standard deviation (409 psi) was computed by taking average of the squares of the strengths, subtracting the average strength squared, and extracting the square root [Eq. (3)]. The strengths and the squares of the strengths can be added on most calculators in one operation.

The coefficient of variation (11.8 percent) is found by computing the standard deviation as a percentage of the average strength. Table 2 indicates that a coefficient of variation of 11.8 percent for average concrete indidates good uniformity.

With an average strength of 3456 psi and a coefficient of variation of 11.8 percent, Fig. 3 shows that nine tests out of ten can be expected to be greater than 85 percent of average or $0.85 \times 3456 = 2940$ psi.

The concrete plant plans to bid on furnishing concrete for two projects: the foundations for a housing project and prestressed concrete for a railroad bridge. The architects for the housing project have specified 2000-psi concrete and will permit a probability of three in ten for tests to fall below that figure. Until sufficient tests have been made to establish a coefficient of variation for this class concrete, 11.8 percent is the best estimate available. From Eq. (7) we find that for an expected coefficient of variation of 11.8 percent, and a value of t from Table 4 of 0.524, the required strength equals

$$f_{cr} = \frac{f_c'}{(1 - tV)} = \frac{2000}{(1 - 0.524 \times 0.118)} = 2130 \text{ psi}$$

The railroad has specified 4000-psi concrete with a probability of no more than one in ten that a test will fall below that figure. The required strength in this case will equal

$$f_{cr} = \frac{4000}{(1 - 1.282 \times 0.118)} = 4710 \text{ psi}$$

Previous strength data from field tests or trial mixes will show water-cement ratios and cement factors required to produce required strengths.

The within-test coefficient of variation (7.1 percent) is computed by multiplying the average range $\overline{R}$ by the factor $1/d_2$ for two companion cylinders, dividing by the average strength, and multiplying by 100. Table 2 indicates that a coefficient of 7.1 percent denotes poor testing practices.

Example 2

To emphasize the difficulties of the consumer, assume that during construction of the bridge, performance tests made periodically at the job site

showed considerably more variation than was anticipated from the mixing plant data. Investigation revealed that the contractor's foreman at times added water to the concrete to facilitate placing operations. Since variation in water-cement ratio is one of the greatest sources of variation of concrete strength, adding water to the plant-mixed concrete increased the coefficient of variation from 11.8 to 19.5 percent, which is listed as fair control in Table 2, and the required strength f_{cr} must be increased to meet design requirements.

$$f_{cr} = \frac{4000}{(1 - 1.282 \times 0.195)} = 5330 \text{ psi}$$

This illustrates the cost of permitting poor control practices since concrete with good control as shown in Example 1 would serve the purpose equally well, with an average strength of 4710 psi instead of 5330 psi.

Example 3

A laboratory desired to check the uniformity of its operations. Table 7 represents test data taken to make this evaluation. Regular test samples were taken in the field and two companion cylinders made from each sample. The average range between companion cylinders (135 psi) and the average strength of all tests (3450 psi) are computed as shown below Table 7. The standard deviation (120 psi) is simply the average range multiplied by the constant established for two cylinders [Eq. (5)]. The coefficient of variation (3.5 percent, within-test) is the standard deviation expressed as a percentage of the average strength. Table 2 indicates that a coefficient of variation of 3.5 percent is indicative of excellent control for field testing procedures and is good for concrete mixed under laboratory controlled conditions. If the equipment and testing methods are accurate the laboratory will provide reliable data on the strength of concrete.

TABLE 7—EVALUATION OF TESTING METHODS

Sample No.	28-day strength, psi		Range, psi	Sample No.	28-day strength, psi		Range, psi
	Cylinder 1	Cylinder 2			Cylinder 1	Cylinder 2	
1	3540	3630	90	10	2990	3260	270
2	3410	3600	190	11	3040	2990	50
3	3420	3450	30	12	3210	3070	140
4	2920	3210	290	13	3810	3770	40
5	3330	3180	150	14	3950	4140	190
6	3730	3560	170	15	3150	3280	130
7	4020	3890	130	16	2950	2970	20
8	3650	3580	70	17	3610	3430	180
9	2830	3030	200	18	3870	3780	90

$\Sigma X = 124{,}150$
$\Sigma R = 2430$
$\bar{X} = \Sigma X/n = 3449$

$\bar{R} = \Sigma R/n = 135$
$\sigma_1 = 0.8865 \times 135 = 120$
$V_1 = (120 \times 100)/3449 = 3.5$ percent

Example 4

Examination of the quality-control charts (Fig. 4) shows that even though the average strength is slightly lower than that required the concrete meets

suggested limits for structural concrete. Some cylinders fall below f_c', but nearly all are close to that value as would be expected from Fig. 1. This gives more confidence in the specified design strength f_c' even though some tests are lower. In addition, the large number of tests whose strengths are considerably in excess of f_c' should also be noted since there may be some compensation in load distribution for a possible low strength test.

The moving average for strength did not fall below the lower limit but whenever the moving average fell below the required average, the number of low strengths for that period increased.

The moving average for range should not vary significantly above the maximum average ($\overline{R}_m$) or the testing must be considered poor.

The control charts indicate the following:

1. The W/C ratio should be lowered to increase the average strength or if possible the coefficient of variation should be reduced to lower the required strength.

2. Control in general is good but the strength varies from high to low as shown by the moving average. An effort should be made to trace the causes of these cycles.

3. The uniformity of testing should be improved to bring the average range down to the allowable average.

Discussion of this paper should reach ACI headquarters in triplicate by Oct. 1, 1957, for publication in the March 1958 JOURNAL.

Under-Reinforced Concrete Beams Under Long-Term Loads*

By HERBERT A. SAWYER, JR.† and JACK E. STEPHENS‡

SYNOPSIS

Effect of an increase in the time of application of load on both deformations and strength of under-reinforced concrete beams is reported. Intermediate grade steel and concrete of 2500 psi and 5000 psi nominal strength were used for the 14 test beams loaded with gravity-type loads. Increase in term of loading had slight effect on ultimate strength. All test strengths, regardless of term of loading, exceeded strengths calculated by the common ultimate load theories. Increase in term of loading caused important modifications in the moment-curvature relationship, including large increases in concrete strain and beam curvature at ultimate load.

PURPOSE AND PROCEDURE

The purpose of these tests was to obtain the following information for under-reinforced concrete beams under long-term loads and under loads of the usual test duration: (1) ultimate strength; (2) concrete strain and beam curvature at ultimate load; and (3) moment-curvature relationship.

Although columns under long-term loads have been investigated,[1] and limited tests of beams under long-term loads have been performed,[2,3] evidently there have been no tests comparing strengths and strains of beams of differing terms of loading but of otherwise "identical" histories, nor have moment-curvature relationships been obtained for long-term-loaded beams.

The tests were performed in three series; procedure for the first two series was designed to eliminate the effect of all variables except the term of loading. Also, comparison of the two series was intended to show the effect of variation in concrete strength on the term-of-loading effect, if any. Thus, for the first and second series, test beams were cast in pairs made as identical as possible. After curing, one beam of each pair was loaded with gradually increasing long-term loads (2 percent increments every fourth day) in the range above 60 percent of the estimated ultimate load until failure occurred. At time of failure the second beam of the pair was tested to failure under short-term loads. The first series consisted of six-beams (three pairs) made of concrete designed for a minimum 28-day strength of 5000 psi. The second series con-

*Received by the Institute Aug. 3, 1956. Title No. 54-2 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 1, July 1957, *Proceedings* V. 54. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Oct. 1, 1957. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

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sisted of six beams (three pairs) made of concrete designed for a minimum 28-day strength of 2500 psi.

The third series was designed to eliminate the effect of all variables except the degree of restraint of the reinforcing steel by the surrounding concrete. This series consisted of two beams of 5000-psi concrete, both tested under long-term loads. The beams were identical except that Beam XL had its reinforcement exposed in its middle third, and Beam L had normal covered reinforcement.*

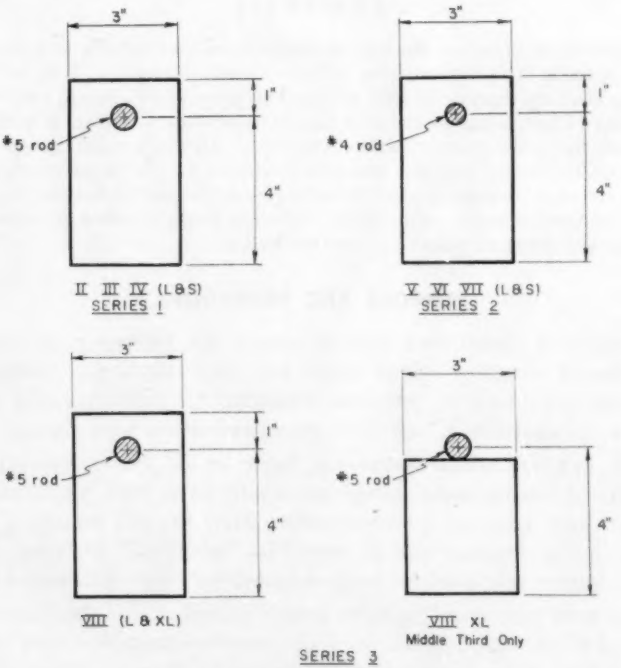


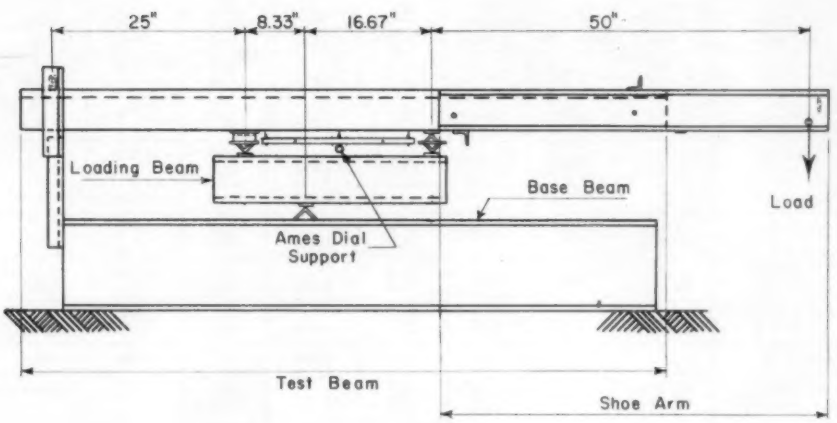
Fig. 1—Cross sections of test beams

All beams were under-reinforced according to the Whitney theory[4] of ultimate strengths. Beam cross sections are shown in Fig. 1. All beams were 84 in. long and contained intermediate grade reinforcing bars with deformations conforming to ASTM A 305-50T.

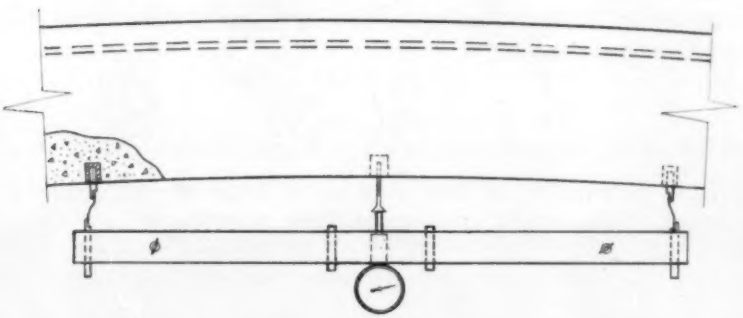
By means of the specially designed apparatus shown in Fig. 2, pure moment was applied with a mechanical advantage to the middle third of the beams by a gravity load, with the resulting dependability of load maintenance inherent in a gravity system.

Both curvature and the total strain in the outer compressive fiber were measured over a 20-in. gage length within the middle third of the beam.

*A detailed report of the investigation discussed herein has been published as a University of Connecticut Engineering Experiment Station bulletin.[6]



(a) Elevation



(b) Detail of Gage Support

Fig. 2—Test apparatus

RESULTS AND INTERPRETATION OF RESULTS

Ultimate strengths

Table 1 summarizes the observed strengths of test beams loaded with short-term and long-term loads, and shows that the strength variation caused by variation in term of loading for under-reinforced beams is not large enough to be significant. Also, this strength variation does not appear to be sensitive to variation in the strength of the concrete in the beam.

TABLE 1—ULTIMATE STRENGTHS*

Series and beam	Time under load	f_c' at 28 days, ksi	f_c' at failure, ksi	M_u observed, in.-k	$\frac{M_{ul}}{M_{us}}$	M_{ul} in.-k	$\frac{M_{us}}{M_{ut}}$	$\frac{M_{ul}}{M_{ut}}$
1—II S	41 min	5.97	7.04	55.2		51.4	1.074	
L	50 days			58.1	1.052	51.4		1.130
1—III S	33 min	5.74	5.92	52.9		52.9	1.000	
L	42 days			53.8	1.019	53.1		1.014
1—IV S	27 min	5.53	5.65	51.1		50.9	1.005	
L	32 days			52.2	1.021	50.9		1.026
1—Avg					1.031		1.026	1.057
2—V S	19 min	2.69	3.10	33.4		30.6	1.092	
L	80 days			34.0	1.018	30.6		1.111
2—VI S	24 min	2.20	2.90	34.5		31.5	1.094	
L	75 days			33.4	0.968	31.5		1.060
2—VII S	35 min	2.60	3.04	33.8		31.7	1.066	
L	75 days			33.1	0.979	31.7		1.044
2—Avg					0.988		1.084	1.072
3—VIII L	62 days	5.64	7.51	60.8		53.7		1.132
VIII XL	53 days			58.2		(53.7)		

*M_u = ultimate moment
M_{ul} = ultimate moment, long-term loading
M_{us} = ultimate moment, short-term loading
M_{ut} = ultimate moment, theoretical

Comparison with ultimate load theory

The theoretical ultimate strengths of the test beams of Table 1 were obtained by the following expression:[4]

$$M_{ut} = T_y\left(d - \frac{T_y}{2f_c'b}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where T_y is the force in the reinforcing bar at its lower yield point. Every beam tested either equaled or exceeded these strengths, the average excess being 6.3 percent.

The following factors are believed to have caused this excess strength:

1. Three of the beams showed evidence of strain-hardening in the steel prior to ultimate moment.
2. The reinforcement, unlike that in practically all beams cited in support of ultimate load theories, had deformations conforming to ASTM A 305-50T.
3. The size effect, not important for the usual short-term tests, tends to increase in importance with increase in term of loading, since the corresponding increase in curvature increases the geometric restraints to compressive failure.
4. The test beams were cast with compression face down.

These factors discourage the conclusion that the common ultimate load theories are overly-conservative. These tests do offer evidence that the ultimate load theory may be extended to concretes in the 6000 to 7000-psi range. Few beam tests previously cited have involved concretes in this range.

Strains in concrete at ultimate load

In Table 2 are listed the first observed strains in the outer fiber of the concrete at ultimate load on the test beams, ϵ_u, averaged over a 20-in. gage length.

TABLE 2—CONCRETE STRAIN AT ULTIMATE LOAD

Series and beam	Time under load, min	f_c' at failure, ksi	ϵ_u, observed, in. per in.	ϵ_u by Eq. (2), in. per in.	$\frac{\epsilon_u \text{ (observed)}}{\epsilon_u \text{ (calculated)}}$
1—II S	41	7.04	0.00227	0.00246	0.923
L	72,000		0.00605	0.00595	1.017
1—III S	33	5.92	0.00220	0.00204	1.078
L	60,800		0.00470	0.00511	0.920
1—IV S	27	5.65	0.00200	0.00188	1.064
L	46,300		0.00480	0.00482	0.996
2—V S	19	3.10	0.00103	0.00113	0.912
L	115,000		0.00308	0.00335	0.919
2—VI S	24	2.90	0.00120	0.00114	1.053
L	108,000		0.00320	0.00320	1.000
2—VII S	35	3.04	0.00110	0.00127	0.866
L	108,000		0.00368	0.00330	1.115
3—VIII L	86,400	7.51	0.00632	0.00637	0.994

It was found that ϵ_u may be expressed as a function of the time under load and the cylinder strength of the concrete by the following equation:

$$1000\,\epsilon_u = \left(0.2 + \frac{f_c'}{8}\right)(\log_{10} t + 0.65) \qquad (2)$$

where f_c' is the cylinder strength in ksi at time of failure, and t is the time under load in min. Table 2 shows strains calculated from this equation. Fig. 3

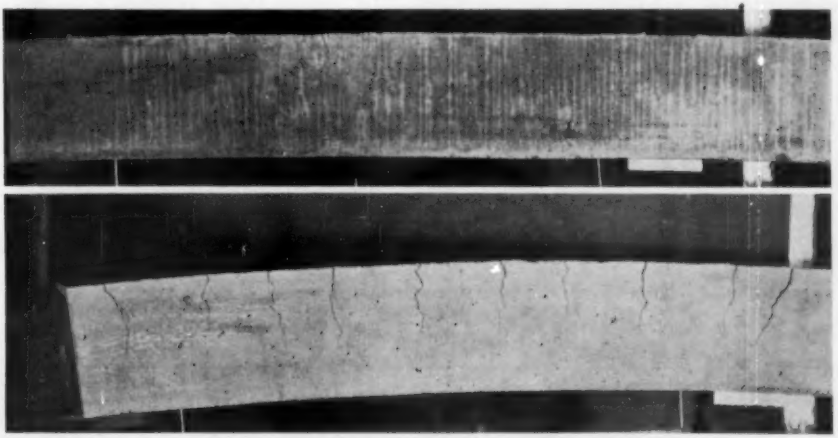


Fig. 3 (top)—Beam II S at 96.5 percent of ultimate load

Fig. 4 (bottom)—Beam II L at 98 percent of ultimate load

and 4 show Beams II S and II L at their near-ultimate concrete strains and curvatures.

Ultimate curvatures

In Table 3 are listed the observed values of ϕ_u, the curvature of the beam at attainment of ultimate load. From this definition it is seen that this curvature occurs almost simultaneously with the concrete strain ϵ_u (Table 2), suggesting that the expression previously derived for ϵ_u may be useful in expressing ϕ_u. It was found that the following expression could be used to express ultimate curvatures:

$$\phi_u = Q \frac{\epsilon_u f_c' b}{T_y} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

where Q tends to be constant, with a value for these tests of 0.60. However, it was found that Eq. (3) will better conform to these test results if Q is given the following value:

$$Q = (0.14 - 0.0006 \log_{10} t)(56 - f'_{ca}) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

where f'_{ca} is the average cylinder strength in ksi for the loading period, and t, as previously defined, is in min. Values of ϕ_u calculated from Eq. (3) and (4) are listed in Table 3. In general, increase in the term of loading from roughly 30 min to 50 days for the beams tested approximately doubled the curvature at ultimate moment.

These semi-empirical expressions for the concrete strain and the curvature at attainment of ultimate load, Eq. (2), (3), and (4), are complex because of the time factor, and they should be regarded as approximations with some experimental verification of the form into which these relationships may eventually evolve as more data accumulates.

TABLE 3—CURVATURE AT ULTIMATE LOAD

Series and beam	T_y (or $A_s f_y$), kips	f'_{ca}*, ksi	ϵ_u by Eq. (2), in. per in.	ϕ_u by Eq. (3), (4), radians per in.	Observed ϕ_u, radians per in.	$\frac{\phi_u \text{ (observed)}}{\phi_u \text{ (calculated)}}$
1—II S	13.9	7.04	0.00246	0.00242	0.00238	0.985
L		6.50	0.00595	0.00459	0.00460	0.998
1—III S	14.7	5.92	0.00204	0.00162	0.00180	1.109
L		5.83	0.00511	0.00342	0.00324	0.947
1—IV S	14.1	5.65	0.00188	0.00151	0.00160	1.059
L		5.59	0.00482	0.00325	0.00300	0.925
2—V S	8.88	3.10	0.00113	0.00083	0.00075	0.901
L		2.90	0.00335	0.00193	0.00213	1.103
2—VI S	8.90	2.90	0.00114	0.00079	0.00084	1.063
L		2.55	0.00320	0.00162	0.00154	0.951
2—VII S	8.92	3.04	0.00127	0.00090	0.00088	0.978
L		2.82	0.00330	0.00184	0.00184	1.000
3—VIII L	14.6	6.58	0.00637	0.00472	0.00452	0.957

*f'_{ca} = average cylinder strength for the loading period

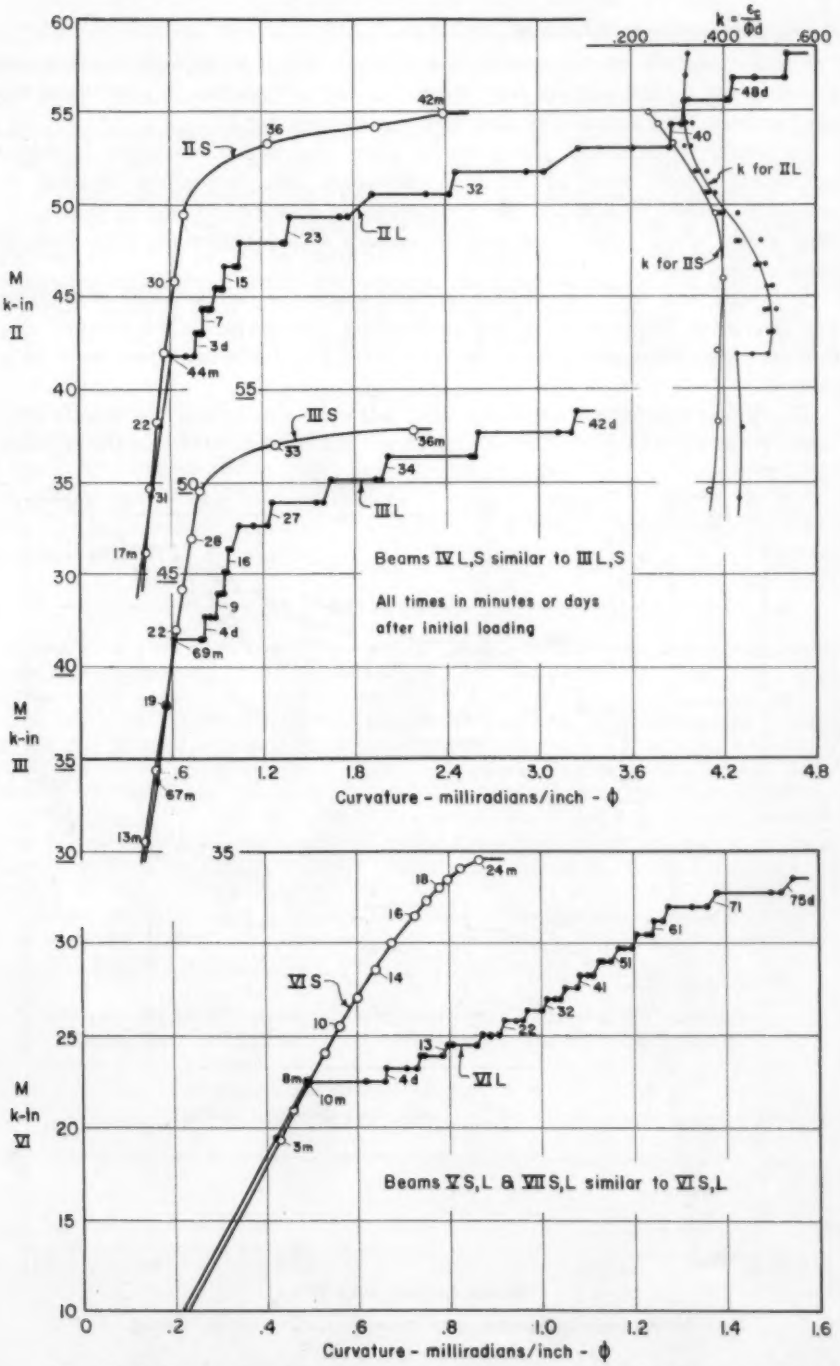


Fig. 5—Typical moment-curvature and moment-k curves, Series 1 and 2

Moment-curvature relationship

Fig. 5 shows the variation of curvature with respect to applied moment and and time for typical pairs of test beams (as well as variation in position of the neutral axis for Beams II S and II L, as defined by k).

As would be expected, these curves show that beam curvature increases appreciably with term of loading. However, the immediate increase in curvature caused by addition of an increment of a long-term load is, in general, that given by the elastic or tangent moduli of the materials, regardless of magnitude of load.

Although time and the bending moment about the primary axis of bending are the most important factors influencing curvature, other factors also influence curvature and should be considered in comparing these tests with others.

The writer has shown elsewhere[5] that both the elastic and the plastic portions of the moment-curvature relationship must be used to obtain the collapse

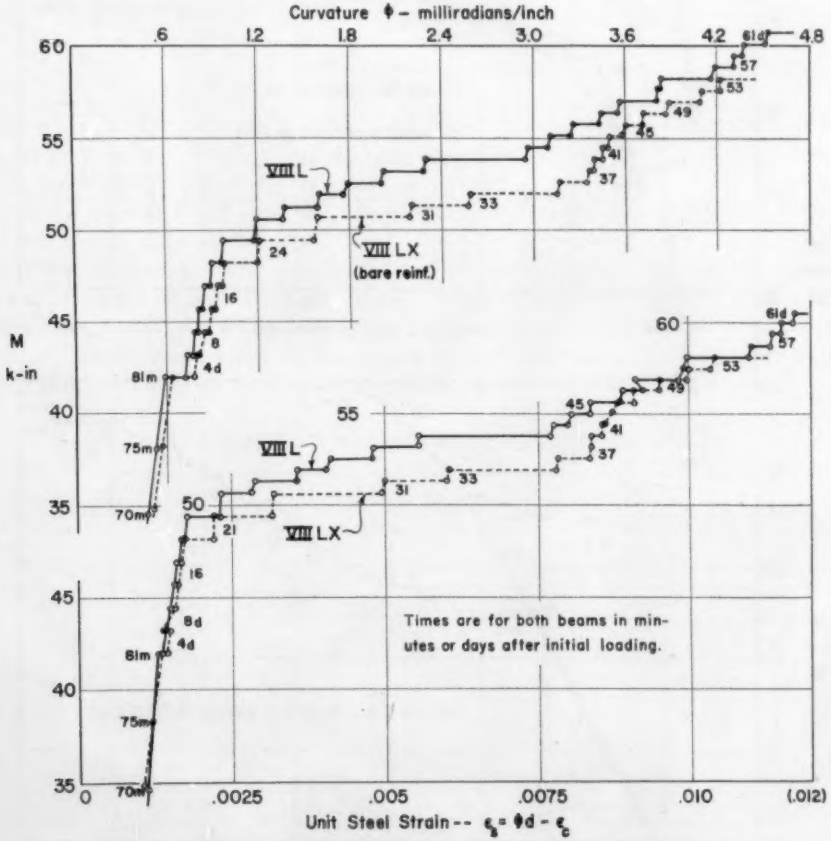


Fig. 6—Moment-curvature and moment-strain curves, Series 3

load for a structure. In the more complete account of these tests[6] semi-rational expressions for the determination of such moment-curvature relationships have been developed.

Steel strains and restraints of tensile concrete

Results of beam tests of Series 3 are shown in Fig. 6. These results show that: (a) At strains below the elastic limit, time increases in curvature are caused entirely by flow of concrete; there is no creep in the steel in this range. (The curves for k of Beams II L and II S, Fig. 5, further verify this conclusion.) (b) The restraint of the tensile concrete on the surrounding reinforcement has but minor effect on steel strain, curvature, and ultimate strength. (Of course tensile concrete is vitally important if shear is present.)

ACKNOWLEDGMENTS

The support of this investigation by the Department of Civil Engineering of the University of Connecticut and the suggestions and encouragement of the department head, Prof. Kenneth C. Tippy, are gratefully acknowledged. Also, this project owes much to Prof. Edwin S. Alling for his participation in the laboratory work involved.

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Discussion of this paper should reach ACI headquarters in triplicate by Oct. 1, 1957, for publication in the March 1958 JOURNAL.

General Method for Analysis of Flat Slabs and Plates*

By JOHN F. BROTCHIE†

SYNOPSIS

A theory for analysis of moments, shears, and deflections in plates is outlined, and a method developed for its application to flat slab and flat plate structures. The slab is considered as floating on the surface of a liquid; each load and each reaction is applied separately to the floating slab, and these separate effects are summed to give actual moments, shears, and deflections in the structure. Theory is applicable to panels of any shape—square, rectangular, triangular, or trapezoidal—with any type of loading. The method allows for any degree of restraint or fixity between slab and column.

Application of the theory in this paper is limited to internal panels with known column reactions, and tables are presented for the quick determination of moment, shear, and deflection at any point of the panel.

NOTATIONS AND DEFINITIONS

Flat slab = Any slab which transmits its loads directly to supporting columns without the use of beams

Flat plate = Special type of flat slab in which, for simplicity, column heads are omitted, and the slab is of uniform thickness throughout

M_x = Bending moment‡ at any point in XX direction (flat slab structure)

M_y = Bending moment at any point in YY direction (flat slab structure)

Q_x = Shear force at any point across a vertical plane YY (flat slab structure)

Q_y = Shear force at any point across a vertical plane XX (flat slab structure)

Δ = Deflection at any point measured from surface of imaginary supporting liquid

M_r = Radial moment caused by a single load or reaction

M_t = Tangential moment caused by a single load or reaction

Q = Shear force at any point caused by a single load or reaction

w = Deflection at any point caused by a single load or reaction

m_r = Radial bending moment at any point caused by unit concentrated load on an infinite floating slab

m_t = Tangential bending moment at any point caused by unit concentrated load on an infinite floating slab

u_1 = Deflection coefficient for unit point load on floating slab

v_1 = Shear coefficient for unit point load on floating slab

*Received by the Institute Dec. 5, 1955. Title No. 54-3 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 1, July 1957, *Proceedings* V. 54. Separate prints are available at 60 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Oct. 1, 1957. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

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‡Bending moments and shear forces are for a unit width of slab.

s = An arbitrarily chosen length selected as a proportion of the column spacing, [Note $s = (D/\gamma)^{1/4}$ = the radius of relative stiffness of the hypothetical floating slab.]*

r = Radial polar coordinate

x = r/s (a dimensionless radial polar coordinate)

θ = Angular polar coordinate, measured from positive X axis

P = Column load or reaction

b = Radius of column head in terms of s

μ = Poisson's ratio (assumed to be 0 for calculation of tables, for any other value of μ: $M_{x\mu} = M_{x_o} + \mu M_{y_o}$

D = Stiffness of slab = $Et^3/12(1 - \mu^2)$ where t = thickness of slab and E = Young's modulus

γ = Density of imaginary supporting liquid (γ is fixed automatically by choosing s)

Σ = Summation sign used to indicate sum of moments, shears, and deflections caused by individual columns and loads

Sign Convention

Loads.....downward positive
Deflections.....downward positive
Moments causing tension on underside of slab are positive.

THEORY

The conventional elastic analysis of a flat slab structure is complicated, often to a degree which makes the analysis impracticable, by the interaction of loads and reactions. As the loads and reactions themselves are generally simple in form or can be divided into simple components, the complexities could be eliminated by indirectly segregating these components.

This can be done by placing an imaginary supporting medium, such as a liquid, against the slab—to enable each load and reaction component to be applied separately. A liquid is a convenient medium as the reaction it provides is proportional to the deflection of the slab. Deflections are measured from the free surface level of the liquid. The reaction of the liquid can be either positive or negative, as the liquid is assumed to always remain in contact with the slab.

Consider an internal panel of a flat slab structure in which the slab is of uniform thickness, elastic, and infinite in extent. Imagine the liquid to be placed beneath, just touching the underside of the slab. The column reactions are considered as applied external forces. Hence if all the loads, including dead loads and their corresponding column reactions, are removed from the slab it will be left weightless and floating on the surface of the liquid.

When a single load is applied, the slab deflects into the liquid causing it to exert a reaction on the slab for which moments, shears, and deflections can be determined. For an infinite floating slab two cases have been tabulated:

*The term s has been chosen to represent the hypothetical radius of relative stiffness of the floating slab to avoid confusion with l, the radius of relative stiffness for a pavement slab. s is merely an arbitrary length while l for a pavement slab is determined by the slab stiffness and subgrade modulus. However, the method is directly applicable to the design of flat slab raft foundations in which case γ is the actual modulus of subgrade reaction of the foundation soil.

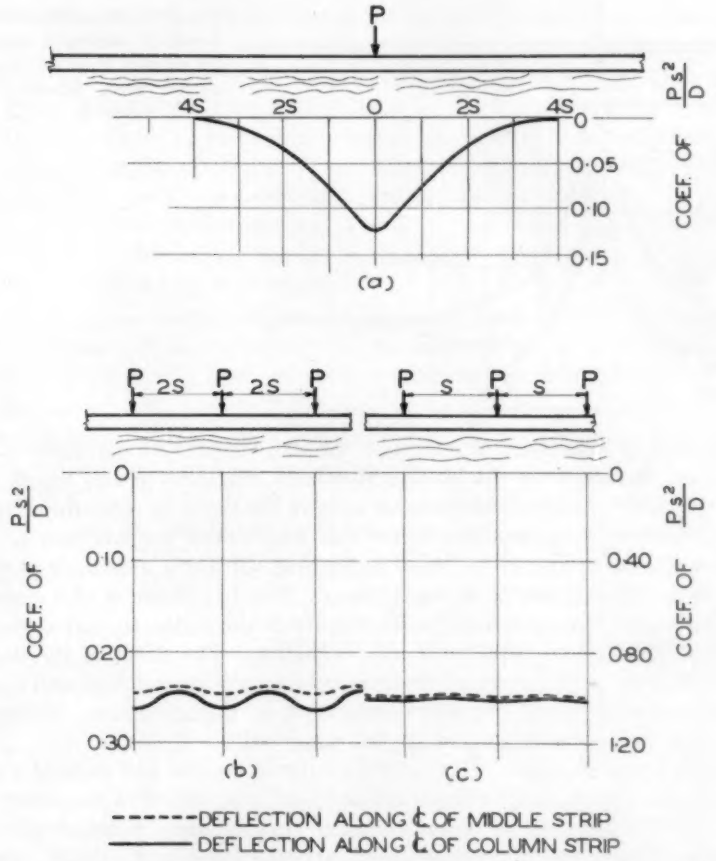


Fig. 1—Deflection of elastic slab floating on liquid under (a) single point load; (b) point loads spaced 2 *s* apart in each direction; and (c) point loads spaced a distance of *s* apart in each direction. Deflection = coefficient $\times$ $P s^2/D$

(1) a single vertical point load* applied to the slab (Tables 1 through 4 and Fig. 1(a) and 2; and (2) a uniform radial moment applied around a circle of small radius (Table 5 and Fig. 3). By combining these two, or by integration from the point loading case, the effect of any type of load can be determined. Hence each load and each column reaction can be applied separately to the floating slab and the moments for each determined from the tables. The sum of these moments gives the moments in the structure when all the loads and reactions act together. When the loads and corresponding column reactions act together however, they form a balanced system of forces so that the

*The case of a point load on an infinite floating slab was first considered in 1884 by the physicist Herts. His original analysis is presented in Reference 1.

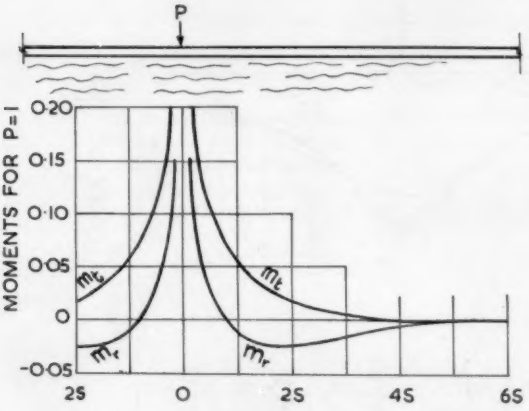


Fig. 2—Tangential and radial bending moments produced by a point load on an infinite elastic slab, floating on a liquid ($\mu = 0$)

net reaction of the liquid is zero, and the slab remains at the surface. The deflections produced by the loads and column reactions acting together will cause the slab to be above the original level of the liquid in parts (in the region of the columns) and elsewhere below this level. The positive and negative liquid reactions produced by these deflections will form a secondary system of forces which will also be in equilibrium. The magnitude of this secondary system of forces is proportional to the density of the liquid, so that if the density is sufficiently low these forces are negligible. The effect of the liquid is thus eliminated, and the sum of the moments caused by each load and reaction acting separately gives the actual moments in the structure. Shears and deflections may be determined in the same way.

A similar procedure can be adopted for external panels, and slabs of varying thickness, in fact for any slab no matter how it is loaded or supported.

For the infinite slab, in order to reduce each separate calculation to two dimensions, polar coordinates are used, and the loads and column reactions divided into symmetrical components if they are not already symmetrical in themselves. The term "symmetrical" is used throughout the paper to indicate polar symmetry, i.e., uniformity around a circle of any given radius, about the center of the load.

Rather than consider the density of the liquid, a parameter s is introduced, such that $s = (D/\gamma)^{1/4}$ where D is the stiffness or flexural rigidity of the slab, and γ the density of the liquid. Use of s eliminates D and γ from the calculations.

All plan dimensions are measured in terms of s. Hence any plan dimension equals xs where x is a dimensionless variable on which moments, shears, and deflections in the floating slab are dependent.

Fig. 1 shows the effect on deflections of varying s. As the liquid is an imaginary one, s can be chosen arbitrarily. Choosing a large value of s merely means choosing a liquid of low density and vice versa. The larger s is chosen in relation to the column spacing, the smaller is the variation in the liquid

reaction under the structure and hence the greater the accuracy of the calculations. If s were chosen equal to ten times the column spacing, the error in moments for a uniformly loaded square panel would be less than 0.5×10^{-5} percent of the maximum moments.

In the limit where s approaches infinity (i.e. $\gamma \to 0$), the moments are the exact moments in the structure. The volume of calculations also approaches infinity however, as it must for exact conventional analysis. By choosing a finite value of s the calculations can be reduced to a reasonably small volume, while retaining any desired degree of accuracy. The volume of necessary calculations is shown in the example.

In summing the moments or shears produced by the individual loads and reactions, the direction θ in which they act must be considered. This can be done by first converting from polar to rectangular coordinates.

In the case of deflections, the direction has no importance; only the distance x of the column or load from the point considered, is required. x and θ are the polar coordinates and are shown in Fig. 4.

Hence if XX and YY are any arbitrarily chosen rectangular axes the actual (total) moments M_x and M_y at any point in the slab are, from simple geometry:

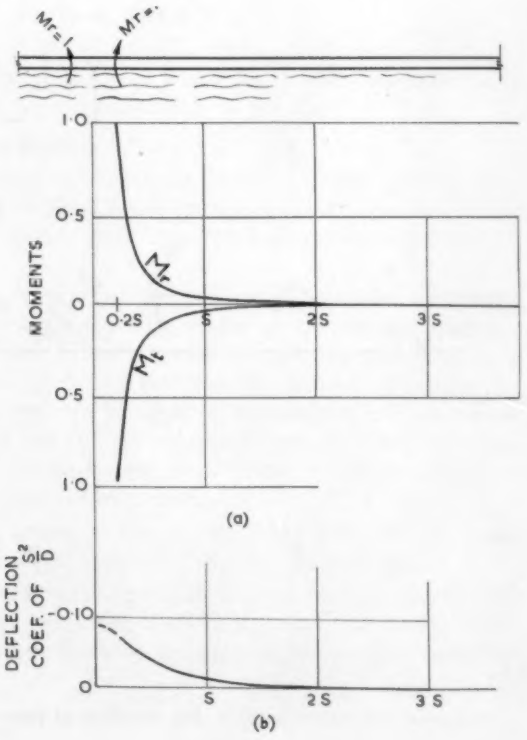


Fig. 3—Radial and tangential moments (a) and deflection (b) when unit moment is applied around a circle of radius 0.2s. (Resistance of portion of plate inside circle is neglected for simplicity.)
$\mu = 0$

$$M_x = \Sigma\,(M_t \sin^2\theta + M_r \cos^2\theta) \quad \ldots\ldots (1)$$

$$M_y = \Sigma\,(M_t \cos^2\theta + M_r \sin^2\theta) \quad \ldots\ldots (2)$$

Shears Q_x and Q_y at any point are:

$$Q_x = \Sigma\, Q \cos\theta \quad \ldots\ldots (3)$$

$$Q_y = \Sigma\, Q \sin\theta \quad \ldots\ldots (4)$$

and the deflection at any point (on the floating slab) is:

$$\Delta = \Sigma\, w \quad \ldots\ldots (5)$$

where M_t, M_r, Q, and w are respectively the tangential and radial moments, shear, and deflection at that point produced by one of the column reactions, or loads, on the floating slab. The actual deflection at any point O in the structure $= \Delta_o - \Delta_{column}$. Eq. (1) to (5) apply to the general case of any type of loading on the slab and any method of support.

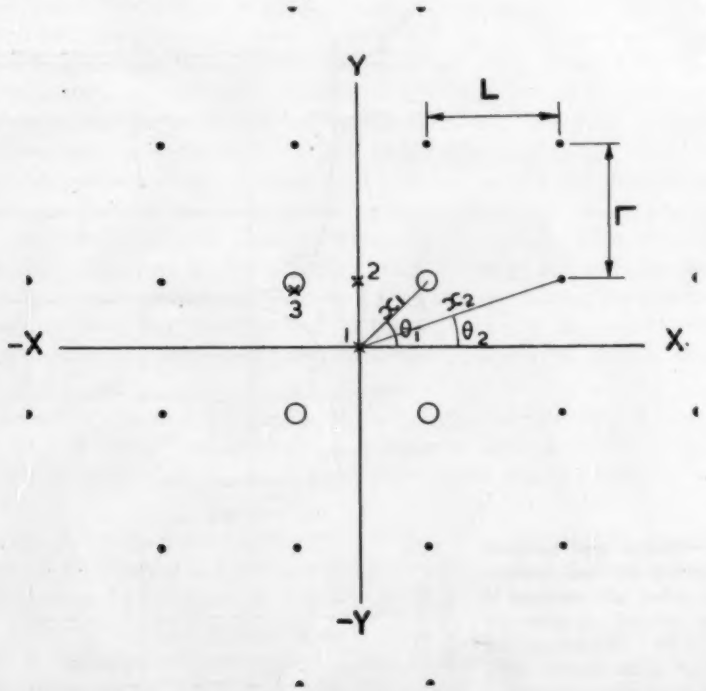


Fig. 4—Plan of slab

For a point load (or reaction) P on the infinite floating slab:

$M_t = m_t P$ in which m_t can be obtained from Table 1
$M_r = m_r P$ in which m_r can be obtained from Table 2
$w = u_1 P s^2/D$ in which u_1 can be obtained from Table 3
$Q = v_1 P/s$ in which v_1 can be obtained from Table 4

Tables 1 to 4 are tabulated from $x = 0$ to $x = 10$, for $4m_t$, $4m_r$, etc. It is convenient to leave them in this form as columns can usually be considered in groups of four, thus slightly simplifying the calculations.

The moments, deflections, and shears produced by a uniform radial moment (ring moment) of magnitude M_{r_b} applied around a circle of radius b where b is small, are given in Table 5. M_t and M_r are as tabulated. $Q = V_o/s$ and $w = U_o s^2/D$. M_t, M_r, V_o, and U_o are tabulated from $x = 0$ to $x = 4$.

Tables 1 to 5 are based on simple elastic plate theory,* and have been calculated using tables of complex functions.[2] Tables 3 and 4 are abridged reproductions from these tables. For convenience Tables 1, 2, and 5 have been calculated for Poisson's ratio $\mu = 0$. Adjustment for other values of μ is simple, and need only to be made to total moments:

$$M_{x\mu} = M_{x_o} + \mu M_{y_o}$$

Tables 3 and 4 are independent of μ, but μ has a slight effect on deflections since it appears in the expression for stiffness D of the slab.

Circular column heads, uniform loading

The application of the theory to uniformly loaded internal panels with circular column heads will now be considered. A uniform load over the whole slab produces no moments or shears, only a uniform deflection, and so need not be considered further.

The effect of a circular column head on the floating slab can be accurately simulated by a point reaction acting at the center of the column, and an applied radial moment acting around the edge (of the column head). This is true for any degree of restraint or fixity between the column and slab, i.e. whether the slab is monolithic with the column, or where there is a horizontal joint between them, or even if the column is penetrating the slab, as in the case of lifted slabs—provided in each case the column reactions are symmetrically distributed around the column head.

The point column reactions acting at the center of the columns are equal in magnitude to the actual column reactions (i.e., the panel loads).

The magnitude of the applied radial moment depends on the size of the column head and the degree of restraint it offers to the slab. It can be determined from any one boundary condition at the edge of the column, radial or tangential moment, or slope.

*A complete version of the mathematics is available from the author on request, together with rigorous determination of the accuracy of the assumptions.

For example if the actual radial moment at the edge of the column is known, or determined, the radial moment caused by the point column reactions can be subtracted to give the (required) applied radial moment. Either the tangential moment or the slope can be used in a similar fashion to determine this applied radial moment. [Slope and tangential moment are related, where the bending is symmetrical: $M_t = -D \times 1/r \times$ slope (where slope = dw/dr).]

TABLE 1—TANGENTIAL MOMENTS, $4 \times m_t$, FOR UNIT POINT LOADS ON FLOATING SLAB

x		0	1	2	3	4	5	6	7	8	9	d
0.0	+	∞	1.66	1.44	1.313	1.220	1.150	1.091	1.043	1.000	0.962	
1	+0.	929	899	872	846	823	801	780	761	743	726	
2	+0.7	095	*943	*795	*657	*521	*392	*269	*152	*036	**928	
3	+0.5	820	719	619	524	429	340	253	168	084	005	
4	+0.4	925	851	776	702	632	564	496	430	367	304	
5		242	182	123	066	009	*955	*900	*847	*795	*744	
6	+0.3	693	644	597	549	503	457	412	369	325	283	
7		240	199	158	119	080	041	003	*966	*929	*892	
8	+0.2	858	822	788	753	720	687	655	622	591	560	
9		529	498	468	439	410	381	353	325	297	270	
1.0		243	217	190	165	139	114	090	065	041	017	− 25
1	+0.1	994	970	947	925	903	881	859	838	816	795	− 22
2	+0.17	742	537	336	138	*944	*748	*556	*362	*172	**984	−196
3	+0.15	800	618	439	263	090	*919	*747	*577	*404	*241	−171
4	+0.14	079	*915	*758	*601	*444	*290	*137	**986	**838	**691	−154
5	+0.12	547	404	257	118	*981	*839	*705	*573	*439	*308	−142
6	+0.11	175	050	*920	*796	*671	*545	*428	*305	*185	*069	−126
7	+0.09	953	836	723	613	500	389	278	175	067	*961	−111
8	+0.08	857	754	653	552	452	354	256	159	064	*969	− 98
9	+0.07	876	784	693	602	513	425	337	251	165	080	− 88
2.0	+0.06	996	914	832	751	671	592	513	436	359	283	− 79
1		208	134	061	*988	*916	*845	*775	*706	*637	*568	− 71
2	+0.05	501	435	369	304	240	176	113	051	*989	*928	− 64
3	+0.04	868	808	749	691	633	576	519	464	409	354	− 57
4		300	246	194	142	090	039	*988	*938	*889	*840	− 51
5	+0.03	792	744	696	650	604	558	513	468	424	380	− 46
6		337	294	252	210	168	128	087	047	007	*968	− 40
7	+0.02	930	892	854	816	780	743	707	671	636	601	− 37
8		566	532	499	465	432	400	367	336	304	273	− 32
9		242	212	182	152	122	094	065	036	008	*981	− 28
3.0	+0.01	953	926	899	873	847	821	795	770	745	720	− 26
1		696	672	648	625	602	578	556	533	511	489	− 24
2		468	446	425	404	384	363	343	323	304	284	− 21
3		265	246	227	209	191	173	155	137	120	103	− 18
4		086	069	053	036	020	004	*988	*973	*957	*942	− 16
5	+0.00	927	913	898	884	869	855	841	828	814	801	− 14
6		788	775	762	749	737	724	712	700	688	676	− 13
7		665	653	642	631	620	609	598	588	578	567	− 11
8		557	547	537	527	518	508	499	490	481	472	− 10
9		463	454	445	437	428	420	412	404	396	388	− 8
4.0		381	373	365	358	351	343	336	329	323	316	− 8
1		309	302	296	290	283	277	271	265	259	253	− 6
2		247	242	236	230	225	220	214	209	204	199	− 5
3		194	189	184	179	175	170	166	161	157	152	− 5
4		148	144	140	136	132	128	124	120	116	113	− 4
5		109	106	102	099	095	092	089	085	082	079	− 3
6		076	073	070	067	064	061	059	056	053	051	− 3
7		048	046	043	041	038	036	034	031	029	027	− 2
8		025	023	021	019	017	015	013	011	009	007	− 2
9		005	+004	+002	+000	−001	−003	−004	−006	−008	−009	− 2
5.	−0.00	010	023	033	041	047	051	054	056	056	056	
6.		055	054	052	050	048	045	042	040	037	034	+ 3
7.		031	029	026	024	022	019	017	016	014	012	+ 3
8.		011	009	008	007	006	005	004	003	003	002	+ 1
9.		002	−001	−001	−000	−000	+000	+000	+001	+001	+001	+ 0

NOTE: m_t = tangential moment at any point due to unit point load on floating slab. Table gives $4m_t$, based on $\mu = 0$.

The boundary conditions at the edge of the column head for various cases are as follows:

1. Where the slab is monolithic with a rigid column head, i.e., the column head fully restraining the slab, the boundary condition at the edge of the column head is: slope = 0 = M_t. This is the case normally assumed in elastic analysis; the matter is further discussed in Appendix A.

2. For the slab resting on the column head (zero restraint) and the reaction concentrated around the edge of the column head, $M_r = M_t$ at the edge of the column head.

TABLE 2—RADIAL MOMENTS, $4 \times m_r$, FOR UNIT POINT LOADS ON FLOATING SLAB

x		0	1	2	3	4	5	6	7	8	9	d
0.0	+	∞		1.124	0.993	903	831	774	724	683	646	
1	+0.	612	582	553	529	505	483	464	444	426	410	
2	+0.3	935	787	645	503	369	248	131	008	*904	*792	
3	+0.2	690	591	501	406	321	230	147	062	*986	*915	
4	+0.1	840	766	698	632	566	501	439	379	319	262	
5	+0.1	207	152	099	047	*997	*947	*900	*853	*807	*762	
6	+0.0	720	677	634	594	554	515	477	439	403	367	
7		334	300	267	234	202	171	141	111	082	055	
8		025	−001	−028	−053	−079	−104	−129	−152	−176	−199	
9	−0.0	221	242	264	285	305	325	345	364	382	400	
1.0		418	436	452	469	485	501	517	531	546	561	− 16
1		575	588	602	616	629	641	653	665	676	688	− 12
2	−0.06	992	*097	*198	*306	*404	*498	*596	*682	*772	*854	− 94
3	−0.07	940	*020	*100	*181	*258	*329	*397	*467	*534	*601	− 71
4	−0.08	660	717	778	835	892	940	987	*037	*086	*131	− 48
5	−0.09	177	222	259	298	331	369	405	433	460	488	− 38
6		515	548	570	596	613	635	658	675	687	709	− 22
7		722	729	741	753	760	769	778	785	787	792	− 9
8		797	794	793	792	792	790	786	779	774	769	+ 2
9		764	757	748	738	728	717	705	692	678	664	+ 11
2.0		648	633	617	600	582	564	544	525	504	483	+ 18
1		462	440	417	393	369	345	320	295	269	242	+ 24
2		215	188	160	132	103	074	044	014	*983	*952	+ 29
3	−0.08	921	889	857	825	792	759	725	692	659	624	+ 33
4		589	554	519	484	448	412	375	339	302	265	+ 36
5		228	191	153	116	078	039	001	*962	*924	*885	+ 39
6	−0.07	846	806	767	728	688	649	608	568	528	488	+ 39
7		448	408	368	326	287	246	205	164	124	083	+ 41
8		041	000	*960	*918	*877	*837	*795	*755	*713	*672	+ 40
9	−0.06	631	590	549	508	466	426	385	343	302	262	+ 40
3.0		220	179	138	098	057	016	*975	*935	*894	*854	+ 41
1	−0.05	814	773	733	693	653	612	573	532	493	453	+ 41
2		414	374	335	295	257	217	178	139	101	062	+ 40
3		023	*985	*946	*908	*870	*833	*794	*756	*719	*682	+ 37
4	−0.04	644	607	571	533	496	460	423	387	350	314	+ 36
5		278	243	207	172	136	101	065	031	*996	*962	+ 35
6	−0.03	928	893	859	825	791	757	724	691	657	624	+ 34
7		592	558	526	494	462	430	397	366	333	303	+ 32
8		272	241	210	179	149	118	088	059	029	*999	+ 31
9	−0.02	970	940	910	882	853	824	796	768	739	711	+ 29
4.0		684	656	628	601	575	547	520	494	468	442	+ 28
1		415	389	364	339	312	287	263	238	213	188	+ 25
2		164	140	116	091	068	045	021	*998	*975	*952	+ 23
3	−0.01	930	907	884	862	841	818	797	775	754	732	+ 23
4		711	691	670	650	629	609	589	568	548	529	+ 20
5		510	491	471	453	433	415	397	377	359	341	+ 18
6		323	306	288	270	253	236	219	202	185	169	+ 17
7		152	136	120	104	088	072	057	041	026	011	+ 16
8	−0.00	996	981	966	952	937	923	908	894	880	866	+ 14
9		852	839	825	812	799	786	773	760	747	735	+ 13
5.	−0.00	723	605	499	404	319	244	177	−119	−068	−024	
6.	+0.00	013	046	072	094	113	126	137	145	150	153	+ 13
7.		153	153	151	148	143	137	132	126	119	112	− 6
8.		106	098	091	084	078	071	065	058	053	047	− 7
9.		043	037	032	028	025	021	018	014	011	009	− 4

NOTE: m_r = radial moment at any point due to unit point load on floating slab. Table gives $4m_r$, based on $\mu = 0$.

3. Any other known degree of restraint can be directly interpolated between these two.

4. The boundary condition for the slab resting on the column head and a uniform reaction over the area of the column head is $M_r = M_t - P/8\pi$.

5. If the column is penetrating the slab with the slab simply supported around the edge of the column—e.g., dead weight moments in a lifted slab—the radial moment will be zero at the column edge.

These boundary conditions apply to the edge of the column or column head when all the column reactions are acting simultaneously.

Thus to the moments caused by the point column reactions, the moments caused by a radial moment applied around the column heads are added in

TABLE 3—DEFLECTION, POINT LOADS—4 u_1

x		0	1	2	3	4	·5	6	7	8	9	d
0.0	+0.5	000	*999	*997	*993	*989	*984	*978	*971	*963	*955	− 5
1	+0.4	946	936	926	915	904	892	880	867	854	840	− 12
2		826	812	797	782	767	751	735	718	701	684	− 16
3		667	650	632	613	595	577	558	539	520	500	− 18
4		480	461	441	421	400	380	359	339	318	297	− 20
5		275	254	233	211	190	168	146	124	102	080	− 22
6		058	036	014	*991	*969	*946	*924	*901	*879	*856	− 23
7	+0.3	834	811	788	765	743	720	697	674	651	628	− 23
8		606	583	560	537	514	491	469	446	423	400	− 23
9		377	355	332	309	286	264	241	219	196	174	− 22
1.0		151	129	106	084	062	040	018	*995	*973	*951	− 22
1	+0.2	929	907	885	864	842	820	799	777	756	734	− 22
2		713	692	671	649	628	607	587	566	545	524	− 21
3		504	483	463	442	422	402	382	362	342	322	− 20
4		302	283	263	244	224	205	186	167	148	129	− 19
5		110	091	072	054	035	017	*998	*980	*962	*944	− 18
6	+0.1	926	908	891	873	855	838	821	803	786	769	− 17
7		752	735	718	702	685	669	652	636	620	604	− 16
8		588	572	556	540	525	509	494	479	463	448	− 16
9		433	418	404	389	374	360	345	331	317	303	− 14
2.0		289	275	261	247	233	220	206	193	180	166	− 13
1		153	140	127	115	102	089	077	064	052	040	− 13
2	+0.10	277	156	037	*918	*800	*683	*567	*452	*338	*225	−117
3	+0.09	112	001	*890	*780	*672	*564	*456	*350	*245	*140	−108
4	+0.08	037	*934	*832	*731	*631	*531	*433	*335	*238	*142	−100
5	+0.07	047	*953	*859	*766	*675	*583	*493	*404	*315	*227	− 92
6	+0.06	140	053	*968	*883	*799	*716	*633	*551	*470	*390	− 83
7	+0.05	311	232	154	077	000	*925	*849	*775	*701	*629	− 75
8	+0.04	556	485	414	344	274	206	138	070	004	*938	− 68
9	+0.03	872	808	744	680	617	555	494	433	373	313	− 62
3.0		255	196	139	082	025	*969	*914	*859	*805	*752	− 56
1	+0.02	699	647	595	544	493	443	394	345	297	249	− 50
2		202	155	109	063	018	*973	*929	*886	*843	*800	− 45
3	+0.01	758	717	676	635	595	556	517	478	440	402	− 39
4		365	329	292	257	221	186	152	118	085	051	− 35
5		019	*987	*955	*923	*892	*862	*832	*802	*773	*744	− 30
6	+0.00	715	687	659	632	605	578	552	526	500	475	− 27
7		451	426	402	378	355	332	309	287	265	243	− 24
8		222	201	180	160	140	120	101	082	063	044	− 20
9		026	+008	−009	−027	−044	−060	−077	−093	−109	−125	− 16
4.	−0.00	140	279	394	488	562	619	661	689	707	714	
5.		712	704	689	669	646	619	589	558	526	493	
6.		459	426	394	362	331	301	272	245	219	195	
7.		172	151	131	113	096	081	067	054	043	033	
8.		024	016	009	−002	+003	+007	+011	+014	+017	+019	
9.	+0.00	020	021	022	023	023	023	022	022	021	020	
10.		020										

Deflection of floating slab under unit point load $= u_1 \frac{s^2}{D}$. Table gives 4 u_1.

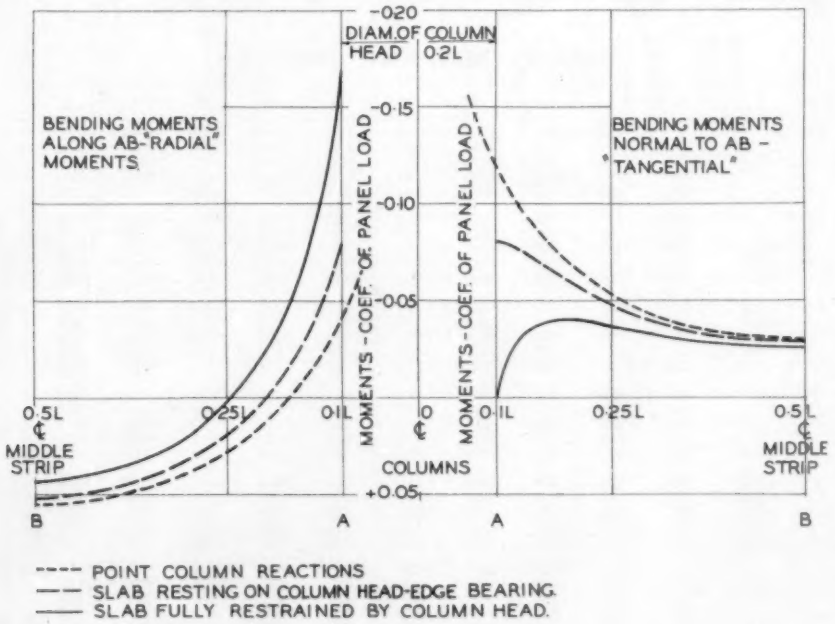
Deflection of floating slab due to point column reactions $= \Delta = \Sigma\, u_1 \frac{P s^2}{D}$

Deflection at any point o in flat slab structure $= \Delta_o - \Delta_{column}$

such a proportion as to produce the required boundary condition. Fig. 5 shows bending moments in a square internal panel with circular column head.

The moments in the portion of the slab within the column head are given in Appendix B.

The liquid has a damping effect on moments, deflections, and shears in the slab, and hence limits the area over which a load is effective. A glance at Fig. 2 will show that the point column reactions likely to affect the moments at a given point are those within a radius of approximately $5s$ (by superposition). For greater accuracy an effective radius x_e has been determined such



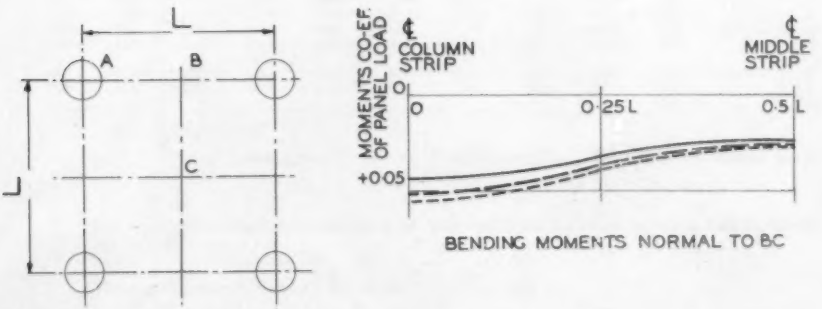


Fig. 5—Bending moments in a square internal panel of a flat slab with circular column heads, radius $0.1L$ ($\mu = 0$)

that the columns outside the circle of this radius produce net moments, shears or deflections of zero at the center. Hence only columns within this circle are included in the calculations. The effective radii for both point column reactions and ring moments are listed in Table 6. For moments due to point column reactions, $x_e = 4.932$. Hence the number of point column reactions to be included in the moment calculations is the number required to support a circle of this radius, i.e.,

$$\text{number of columns} = \frac{\pi\,(4.932)^2}{\text{panel area}} \qquad \left(\begin{matrix}\text{panel area}\\ \text{in terms of } x\end{matrix}\right)$$

TABLE 4—SHEAR, POINT LOADS—4 v_1

x		0	1	2	3	4	5	6	7	8	9	d
0.0	—	∞	63.7	31.8	21.2	15.9	12.7	10.6	9.08	7.94	7.05	+ 32
1		6.34	5.76	5.28	4.86	4.51	4.21	3.94	3.70	3.49	3.30	+ 30
2	−3.	134	*980	*840	*712	*594	*486	*385	*292	*206	*125	+108
3	−2.	050	*979	*913	*850	*791	*736	*683	*633	*585	*540	+ 55
4	−1.	497	456	417	380	344	310	277	246	216	186	+ 34
5		159	132	106	081	056	033	010	*989	*968	*947	+ 23
6	−0.	927	908	889	871	854	837	820	804	788	773	+ 17
7	−0.7	582	437	296	159	025	*894	*766	*642	*520	*402	+131
8	−0.6	286	172	061	*953	*847	*744	*642	*543	*446	*351	+103
9	−0.5	258	166	077	*989	*903	*819	*737	*656	*576	*498	+ 84
1.0	−0.4	422	347	273	201	130	060	*992	*925	*859	*794	+ 70
1	−0.3	730	667	606	545	486	427	370	313	257	203	+ 59
2		149	096	044	*993	*942	*893	*844	*796	*749	*702	+ 49
3	−0.2	656	611	567	523	480	438	396	355	314	274	+ 43
4		235	196	158	120	083	047	011	*976	*941	*907	+ 36
5	−0.1	873	839	807	774	742	711	680	649	619	590	+ 31
6		560	532	503	475	448	421	394	367	341	316	+ 28
7		290	265	241	217	193	169	146	123	100	078	+ 24
8		056	035	014	*993	*972	*952	*931	*912	*892	*873	+ 20
9	−0.0	854	835	817	799	781	763	746	729	712	695	+ 18
2.0	−0.06	786	625	466	309	154	002	*852	*704	*558	*414	+152
1	−0.05	273	133	*996	*861	*727	*596	*467	*339	*214	*090	+131
2	−0.03	969	849	731	614	500	387	276	167	059	*953	+113
3	−0.02	849	746	645	545	447	351	256	163	071	*981	+ 96
4	−0.01	892	804	718	633	550	468	387	308	230	153	+ 82
5		078	004	*931	*859	*789	*719	*651	*584	*519	*454	+ 70
6	−0.00	391	328	267	207	148	090	033	+023	+078	+132	+ 58
7	+0.00	185	237	288	338	387	435	482	528	574	618	+ 48
8		662	705	747	788	828	867	906	944	981	*017	+ 39
9	+0.01	053	087	121	155	187	219	250	280	310	339	+ 32
3.	+0.01	367	615	806	945	*041	*100	*127	*127	*103	*061	
4.	+0.02	004	*934	*855	*768	*676	*580	*482	*384	*286	*189	
5.	+0.01	095	003	*915	*830	*750	*673	*601	*534	*471	*413	
6.	+0.00	359	309	263	222	184	150	119	091	067	046	
7.		027	+010	−004	−016	−026	−034	−041	−047	−051	−054	
8.	−0.00	056	057	058	058	057	056	054	052	050	048	
9.		045	043	040	037	035	032	030	027	025	022	
10.		020										

Shear caused by unit load on floating slab $Q_r = \frac{v_1}{s}$. Table gives 4 v_1.

Shear at any point in flat slab structure due to point column reactions—

$$Q_x = \Sigma\, v_1 \frac{P}{s} \cos\theta$$

$$Q_y = \Sigma\, v_1 \frac{P}{s} \sin\theta$$

TABLE 5—MOMENTS, DEFLECTIONS, AND SHEARS CAUSED BY RING MOMENTS*

x	Moments		Deflection	Shear	x	Moments		Deflection	Shear
	M_t	M_r	U_o	V_o		M_t	M_r	U_o	V_o
0.01	−6370	+6370	—	+0.02	0.4	− 3.742	+ 4.200	−0.68	+0.20
2	−1590	+1590	−2.56	+0.03					
3	− 707	+ 707	−2.31	+0.04	0.5	− 2.318	+ 2.745	−0.54	+0.21
4	− 397	+ 397	−2.12	+0.05	0.6	− 1.545	+ 1.951	−0.44	+0.22
					0.7	− 1.083	+ 1.466	−0.36	+0.23
5	− 254	+ 254	−1.98	+0.06	0.8	− 0.786	+ 1.147	−0.29	+0.23
6	− 177	+ 177	−1.87	+0.07	0.9	− 0.584	+ 0.922	−0.23	+0.23
7	− 130	+ 130	−1.77	+0.07					
8	− 99.3	+ 99.8	−1.68	+0.08	1.0	− 0.442	+ 0.757	−0.18	+0.22
9	− 78.3	+ 78.8	−1.61	+0.09	1.1	− 0.339	+ 0.632	−0.14	+0.22
					1.2	− 0.262	+ 0.533	−0.11	+0.21
0.10	− 63.4	+ 63.9	−1.54	+0.09	1.3	− 0.204	+ 0.454	−0.08	+0.21
1	− 52.4	+ 52.9	−1.48	+0.10	1.4	− 0.160	+ 0.390	−0.05	+0.20
2	− 44.0	+ 44.5	−1.43	+0.10					
3	− 37.4	+ 37.9	−1.38	+0.11	1.5	− 0.125	+ 0.336	−0.03	+0.19
4	− 32.2	+ 32.7	−1.33	+0.12	1.6	− 0.098	+ 0.291	−0.02	+0.18
					1.7	− 0.076	+ 0.251	−0.00	+0.17
5	− 28.1	+ 28.6	−1.28	+0.12	1.8	− 0.059	+ 0.218	+0.01	+0.16
6	− 24.6	+ 25.1	−1.24	+0.12	1.9	− 0.045	+ 0.188	+0.02	+0.15
7	− 21.8	+ 22.3	−1.21	+0.13					
8	− 19.4	+ 19.9	−1.17	+0.13	2.0	− 0.034	+ 0.163	+0.03	+0.14
9	− 17.4	+ 17.9	−1.14	+0.14					
					2.2	− 0.018	+ 0.121	+0.04	+0.12
0.20	− 15.67	+ 16.15	−1.10	+0.14					
1	− 14.19	+ 14.67	−1.07	+0.15	2.4	− 0.008	+ 0.088	+0.04	+0.10
2	− 12.91	+ 13.39	−1.04	+0.15					
3	− 11.79	+ 12.27	−1.02	+0.15	2.6	− 0.001	+ 0.062	+0.05	+0.09
4	− 10.81	+ 11.29	−0.99	+0.16					
					2.8	+ 0.002	+ 0.043	+0.04	+0.07
5	− 9.94	+ 10.42	−0.96	+0.16					
6	− 9.17	+ 9.64	−0.94	+0.16	3.0	+ 0.005	+ 0.028	+0.04	+0.06
7	− 8.49	+ 8.96	−0.92	+0.17					
8	− 7.88	+ 8.35	−0.89	+0.17	3.2	+ 0.006	+ 0.016	+0.04	+0.05
9	− 7.33	+ 7.80	−0.87	+0.17					
					3.4	+ 0.006	+ 0.008	+0.04	+0.04
0.30	− 6.833	+ 7.300	−0.85	+0.17					
1	− 6.384	+ 6.849	−0.83	+0.18	3.6	+ 0.006	+ 0.001	+0.03	+0.03
2	− 5.978	+ 6.441	−0.81	+0.18					
3	− 5.606	+ 6.067	−0.79	+0.18	3.8	+ 0.006	− 0.003	+0.03	+0.02
4	− 5.268	+ 5.727	−0.78	+0.18					
					4.0	+ 0.005	− 0.006	+0.02	+0.02
5	− 4.960	+ 5.418	−0.76	+0.19					
6	− 4.675	+ 5.131	−0.74	+0.19					
7	− 4.414	+ 4.868	−0.72	+0.19					
8	− 4.171	+ 4.623	−0.71	+0.19					
9	− 3.949	+ 4.399	−0.69	+0.20					

*Moments, shears, and deflections caused by a radial moment of M_r applied around a circle of radius $x = b$, where b is small. For example, if $b = 0.10$, the table gives the moments, deflections, and shears from $x = b = 0.10$ to $x = 4.0$ for a radial moment M_r of 63.9 at $x = b$. For unit radial moment divide by M_r.

TABLE 6—EFFECTIVE RADIUS, x_e

		x_e	πx_e^2
POINT COLUMN REACTIONS	Moment	4.932	76.42
	Shear*	—	—
	Deflection*	2.666	22.33
		or (7.17)	(162)
RING MOMENTS	Moment	2.666	22.33

Number of columns to be included in calculations = $\dfrac{\pi x_e^2}{\text{panel area}}$

*For shear and deflection, x_e normally is not restricted. The number of columns to be included is dependent only on the degree of accuracy required. Values of x_e for deflection have been included however as they can be used to calculate deflections of flat slab raft foundations, for the design of which this method has direct application.

The supported area should be as nearly circular as possible. In calculating moments due to point reactions, the actual area πx_e^2 should be closely adhered to, even if portions only of the outermost ring of columns are included (see example). For ring moments this is not so important as the moments produced decrease rapidly with increase in x (Fig. 3).

For calculating deflections in the slab any fixed number of columns may be included, as the error involved in using the wrong number of columns is eliminated when the deflection at the column is subtracted from the deflection at the point considered, provided the same number of columns is used for each. For shear also x_e is not restricted. Sufficient columns should be included in each case to limit the effect of nonuniform distribution of the outermost ring of columns.

The shears and deflections caused by the ring moments are small and decrease rapidly as x increases. For approximate calculations they may be neglected. Even for accurate calculations, ring moments need only be applied to the closest columns; the other columns are considered as point reactions.

EXAMPLE

Analysis of a uniformly loaded square internal panel with circular column heads of radius $0.1L$ (Fig. 4) follows. The calculations are repeated several times to show various degrees of accuracy.

Moments caused by point reactions (at column centers)

Moment at Point 1, Fig. 4, center of panel. From Eq. (1) and (2) and Tables 1 and 2.

Accurate analysis: $L = 8$

No. columns, n	x	$4m_t$ ·	$4m_r$	$\cos^2$	$\sin^2$	$nm_t \cos^2$	$nm_r \sin^2$
4	0.7071	+0.3211	+0.0310	½	½		
4	2.121	+0.06054	−0.09415	½	½		
4	3.535	+0.00877	−0.04154	½	½		
		+0.39041	−0.10469		→	+0.19520	−0.05235
8	1.5811	+0.11423	−0.09464	½	½		
8	2.550	+0.03558	−0.08039	½	½		
8	3.535	+0.00877	−0.04154	½	½		
8	4.528	+0.00100	−0.01457	½	½		
8	3.808	+0.00549	−0.03247	½	½		
8	2.915	+0.02198	−0.06570	½	½		
8	4.743	+0.00037	−0.01083	½	½		
8	4.301	+0.00193	−0.01928	½	½		
		+0.18935	−0.35942		→	+0.18335	−0.35942
0.42	4.95	0	−0.00786	½	½	—	−0.00041
76.42					−0.02763 ←	+0.38455	−0.41218

Therefore $M_x = M_y = +0.0276P$, where P is the panel load. Timoshenko[3] obtains the same value by conventional elastic analysis.

Design analysis: $L = 2s$

No. columns, n	x	$4m_t$	$4m_r$	$\cos^2$	$\sin^2$	$nm_t \cos^2$	$nm_r \sin^2$
4	1.414	+0.1385	−0.0874	½	½	+0.0693	−0.0437
4	4.242	+0.0022	−0.0206	½	½	+0.0011	−0.0103
8	3.16	+0.0156	−0.0557	½ (avg.)	½ (avg.)	+0.0156	−0.0557
3.1	5.10	−0.0002	−0.0061	½ (avg.)	½ (avg.)	+0.0001	−0.0024
19.1				−0.0260 ←		+0.0861	−0.1121

Therefore $M_x = M_y = +0.026\ P$. If calculated error due to nonuniform liquid reaction is applied as a correction, $M_x = M_y = +0.027P$.

Approximate analysis: $L = 3s$. Tabulation is unnecessary.

$$M_x = M_y = -P\left[(+0.0606\ -0.0942)\tfrac{1}{2} + \frac{4.5}{4}(+0.0004\ -0.0109)\tfrac{1}{2}\right]$$
$$= +0.023P$$

Adding correction for liquid reaction, $M_x = M_y = +0.026P$. Number of columns included $= \pi\ (4.932)^2/[L/s]^2$ in each case, from Table 6.

Moment at Point 2, Fig. 4, (intersection of center lines of column strip and middle strip)
Design (or accurate) analysis: $L = 2s$. (Errors due to liquid reaction are smaller at this point.)

From tabulation (as before)

$$M_x = +0.055P$$
$$M_y = -0.030P$$

Approximate analysis: $L = 3s$

$$M_x = -P\left[\tfrac{1}{2}(-0.0918\ -0.0151) + (0.0117)\tfrac{4}{5} + (-0.0483)\tfrac{1}{5} + \tfrac{1}{8}(-0.0005)\frac{4}{13} + \tfrac{1}{8}(-0.0030)\frac{9}{13}\right]$$
$$= +0.054P$$
$$M_y = -P[\tfrac{1}{2}(+0.1255\ +0.0011) + (0.0117)\tfrac{1}{5} + (-0.0483)\tfrac{4}{5} + 0.000]$$
$$= -0.027P$$

Moment at Point 3, Fig. 4, (column edge)

Accurate analysis: $L = s$

$$M_x = -0.120P$$
$$M_y = -0.041P$$

Design analysis: $L = 2s$

$$M_x = -0.120P$$
$$M_y = -0.041P$$

"Approximate" analysis: $L = 3s$

Moment due to closest column $M_x = -P\ \tfrac{1}{4}\ (0.582) = -0.146P$
$M_y = -P\ \tfrac{1}{4}\ (0.269) = -0.067P$

Moments due to other columns

$$M_x = M_y = -P\left[\tfrac{1}{2}(+0.020\ -0.062) + \frac{3.5}{4}(+0.002\ -0.021)\tfrac{1}{2}\right]$$
$$= +0.029P$$

Total moments $M_x = -0.117P$
$M_y = -0.038P$

Correcting for liquid reaction, $M_x = -0.120P$; $M_y = -0.041P$. A moment pattern for the whole panel can be plotted easily by first plotting a moment pattern for all except the closest columns. These moments are almost uniform so that only a few points are required. The moments due to the closest columns are then calculated at closer intervals and superimposed on the base pattern.

Moments caused by applied ring moments, radius of column head = 0.1*L*

Design analysis: $L = 2s$. The moments are taken directly from Table 5; their actual magnitude is determined later; i.e., moments are initially determined for an applied radial moment of +16.15 acting around the column heads (radius = $0.2s$). From Table 5 and Eq. (1) and (2):

Moment at Point 1 (center of Panel), Fig. 4, considering 5.58 columns (Table 6)

$$M_x = M_y = 2 \times 0.39 - 2 \times 0.16 + 1.58\,(0.006 + 0.018)\,\tfrac{1}{2} = +0.48$$

Moment at Point 2 (intersection of center lines of column and middle strip) for 5.58 of six closest columns (Table 6).

$$M_x = 2 \times 0.76 + 3.58\,(1/5 \times 0.12 + 4/5 \times -0.018) = +1.55$$
$$M_y = -2 \times 0.44 + 3.58\,(4/5 \times 0.12 + 1/5 \times -0.018) = -0.55$$

Moment at Point 3 (column edge) for the five closest columns, effect of others being negligible (in this case).

Moment due to closest column: $M_x = -15.67 \quad M_y = +16.15$
Moment due to next four columns: $M_x = M_y = 2 \times -0.034 + 2 \times 0.163 = +0.26$
Total moment: $M_x = -15.41, \quad M_y = +16.41$

Total moments—where column head fully restrains slab

Boundary condition at column edge is $M_t = 0$ (i.e. $M_x = 0$ at Point 3). Hence add ring moments (above) to point reaction moments in such a proportion as to produce this condition. Therefore factor by which ring moments should be multiplied is:

$$\frac{+0.120P}{-15.41} = -0.0078P$$

and total moments are:

Point 1—$M_x = M_y = +0.27P - 0.0078 \times 0.48P = +0.023P$
Point 2—$M_x = +0.055P - 0.0078 \times 1.55P = +0.043P$
$M_y = -0.030P - 0.0078 \times -0.55P = -0.026P$
Point 3—$M_x = 0$
$M_y = -0.041 - 0.0078 \times 16.41 = -0.169P$

Total moments—where slab rests on column, edge bearing

Boundary condition at column edge is $M_r = M_t$; therefore $M_x = M_y$. Factor by which ring moments are multiplied is

$$\frac{0.120 - 0.041}{-15.41 - 16.41}\,P = -0.0025P$$

At Point 1—$M_x = M_y = +0.027P - 0.0025 \times 0.48P = +0.026P$
At Point 2—$M_x = +0.055P - 0.0025 \times 1.55P = +0.051P$
$M_y = -0.030P - 0.0025 \times (-0.55)P = -0.029P$
At Point 3—$M_x = -0.120P - 0.0025 \times (-15.41)P = -0.082P$
$M_y = -0.041P - 0.0025 \times 16.41P = -0.082P$

Shear

From Eq. (3) and (4) and Table 4, choosing $L = s$, shear is:

At Point 1—$Q_x = Q_y = 0$ ($\Sigma \cos\theta = 0$)
At Point 2—$Q_x = Q_y = 0$ ($\Sigma \cos\theta = 0$)
At Point 3—$Q_x = 0$ ($\Sigma \cos\theta = 0$)

$$Q_y = \frac{6.34}{4}\frac{P}{s} = 1.59\frac{P}{L}$$

Check by normal method: $Q_y = \dfrac{P}{2\pi \times 0.1\,L} = 1.59\dfrac{P}{L}$

The equations and tables are obviously not necessary for these three points. If shears are required at other points in the panel, however, this method will prove of advantage.

Deflection at center of panel

Using Eq. (5) and Table 3, let $L = 2s$; consider only point column reactions. Any number of columns may be included. (Including $\pi x_c^2/2^2 = 40$, however, gives the actual deflections of the floating slab due to the point reactions. This is included for interest only.)

No. of columns, n	x	$4\,u_1$	$n\,u_1$	
4	1.414	+0.2275	+0.2275	
4	4.242	−0.0044		−0.0044
8	3.162	+0.0238	+0.0476	
8	5.10	−0.0070		−0.0140
8	5.83	−0.0052		−0.0104
8	7.07	−0.0016		−0.0032
40			+0.2751	−0.0320

$+0.2751 - 0.0320 = +0.2431 = \Delta_{center}$

No. of columns, n	x	$4\,u_1$	$n\,u_1$	
1	0	+0.5000	+0.1250	
4	2	+0.1289	+0.1289	
4	4	−0.0014		−0.0014
4	2.828	+0.0436	+0.0436	
8	4.472	−0.0060		−0.0120
4	5.66	−0.0057		−0.0057
4	6.00	−0.0046		−0.0046
8	6.32	−0.0036		−0.0072
3	7.21	−0.0013		−0.0010
40			+0.2975	−0.0319

$+0.2975 - 0.0319 = +0.2656 = \Delta_{columns}$

Therefore deflection at center of panel is $\Delta_{center} - \Delta_{columns}$ or $0.0225\,P\,s^2/D = 0.0056\,PL^2/D$. By conventional elastic analysis, this deflection is $0.0058\,PL^2/D$.

Deflection can be calculated more accurately if more columns are included. For approximate calculations, only the closest columns need be included; e.g., summing six columns:

$$\text{Defl'n} = \left[(0.228 + \tfrac{1}{2} \times 0.024) - (\tfrac{1}{4} \times 0.500 + 0.129 + \tfrac{1}{4}\,0.044)\right]\frac{-Ps^2}{D} = 0.006\,\frac{PL^2}{D}$$

NONUNIFORM LOADS

The case of a concentrated load at the center of each panel could be analyzed in a similar manner to the uniform loading case. The moments caused by the point loads in this case, would be added to the moments caused by the column reactions to give the actual moments in the structure. Any regular loading which produces equal reactions at each column, can be treated in a similar way.

For the general case of nonuniform loading or irregular supports an additional factor involved is the calculation of the column reactions. These also can be determined using the floating slab theory.

SUMMARY

Consider the slab as floating on the surface of a liquid; the column reactions are equal to the slab loading so that the net reaction of the liquid is zero.

Choose s, radius of relative stiffness of the floating slab, in relation to column spacing to provide the required accuracy. For accurate analysis s = column spacing. For design analysis $s = \frac{1}{2}$ column spacing; approximate analysis requires $s = 1/3$ column spacing—or any intermediate value.

All plan dimensions of the slab are measured in terms of s; i.e., any plan dimension equals xs, where x is the variable for which moments, shears, and deflections are tabulated.

Consider each load and reaction as acting separately on the floating slab. The sum of these separate effects represents the actual moments, shears, and deflections in the structure.

To calculate the moments caused by circular columns on the floating slab, replace columns by point reactions, and calculate moments. Add the moments caused by a radial moment applied at the edge of the columns, or column heads, in such a proportion as to produce the required boundary condition at the column edge.

A uniform load over whole slab produces no moments, shears, or differential deflections in the floating slab.

Moments, shears, and deflections caused by point loads (or reactions) on the slab are listed in Tables 1 to 4. Moments, shears, and deflections caused by a radial moment applied around a circle of small radius may be obtained from Table 5.

Tables 1, 2, 4, and 5 give moments and shears with polar coordinates. Eq. (1) and (2) sum moments and transform them to cartesian coordinates. Shears are treated similarly [Eq. (3) and (4)].

Deflections are calculated from Table 3 and Eq. (5). The number of columns to be summed in Eq. (1) to (5) is the number required to support the area πx_e^2 (Table 6). Supported area should be as nearly circular as possible. Use exact number of columns, including portions of outermost columns if necessary.

Thus, using the concept of the floating slab, the moments, shears, and deflections in the structure can be determined with any desired degree of accuracy.

ACKNOWLEDGMENTS

The author wishes to express his indebtedness to M. G. Dempster, assistant director of engineering and W. C. Alexander, chief structural engineer, of the Department of Works, Melbourne, for their interest and support; to A. J. Francis, professor of civil engineering, University of Melbourne, for advice and encouragement; to F. J. Syer and J. P. Ryan, senior lecturers in mathematics at Melbourne University; to G. T. Hunt and fellow engineers in the Department of Works for willing assistance.

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APPENDIX A

Where the column is integral with the slab, the degree of restraint (offered by the column to the slab) must be determined or assumed. For elastic analysis it is usually assumed as 100 percent.[3,4] In actual fact it will be slightly less due to the elasticity of the column or column head.

For a reinforced concrete slab it is reduced still further by tensile cracking around the column face. For accurate analysis, each of these effects can be taken into account, if one boundary condition at the column edge is determined. The slope of the slab allowed by the deformation of the column in one case, or by the cracking of the slab in the other, is the logical one to determine.

For design an accurate determination of the degree of restraint may not be necessary however, for if the loading is increased towards the ultimate flexural load, plastic yielding around the column face will allow the tangential moment to increase, in each case, until it equals the radial moment. This will be considered in more detail in a later paper on plastic analysis.

APPENDIX B

For the case of the slab resting on the column the moments in the slab within the column head may be determined by integration from the point load case. If the reaction between the slab and column is uniform over the area of the column, the moment will be a maximum at the center. This maximum moment is M_o where

$$M_o = P(m_t)_{x=b}$$

in which m_t is obtainable from Table 1.

If the reaction is concentrated round the edge of the column, the moment M_o' is constant over the column head.

$$M_o' = \frac{P}{2}(m_t + m_r)_{x=b}$$

in which m_t and m_r are obtainable from Tables 1 and 2 respectively.

It should be noted that M_o and M_o' are the moments caused by the closest column only. Moments due to other columns and loads must be added to give the total moments in the slab.

If the slab and the column head are cast integrally however, the moment inside the head is not of considerable importance, because of the greater depth available to resist it.

Discussion of this paper should reach ACI headquarters in triplicate by Oct. 1, 1957, for publication in the March 1958 JOURNAL.

Effect of a Waterproof Coating on Concrete Durability*

By WARRINGTON G. MITCHELL†

SYNOPSIS

Common denominator of all generally recognized types of concrete deterioration is water, either in liquid or crystalline form. Since concrete is characteristically a porous material, improvements in concrete itself are not likely to completely prevent ingress of potentially harmful water. Author believes waterproof coatings can speedily and economically improve concrete durability.

Freeze-thaw tests on beams coated with neoprene latex showed significantly greater durability than that of uncoated specimens. Five-hr sand-blast test demonstrated improved abrasion resistance of neoprene-latex coated specimens.

INTRODUCTION

Application of effectually impervious surface coatings to certain parts of concrete structures to prevent passage of water through the concrete has long been an accepted practice. The possibility of extending the use of such coatings to the achievement of more durable concrete, however, has not been thoroughly comprehended, nor evaluated. Anyone concerned with concrete durability is well aware of the beneficial effects of increased imperviousness; in fact, the main effort of "preventive research" is in the attainment of a higher degree of impermeability. The author takes no issue with the correctness of the aim of concrete research but he does take issue with the idea of achieving it solely through the improvement of the product, per se. The idea of extending the use of protective coatings to increase concrete durability came to the author as a result of limited tests performed by him in 1948. The principle involved and the test results obtained have not previously been reported in the hope of an opportunity for furthering the initial research. Unfortunately this opportunity has not materialized nor is there any prospect for continuing research by the author in the foreseeable future. The present paper is presented with the expressed hope that it will stimulate thought in formulating a new trend in concrete research.

*Received by the Institute Sept. 14, 1956. Title No. 54-4 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 1, July 1957, *Proceedings* V. 54. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Oct. 1, 1957. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

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DURABILITY RESEARCH

In the introduction to their paper, "Petrographic and Mineralogic Characteristics of Aggregates," Rhoades and Mielenz* make the following observations regarding the complexity of interrelated factors inherent in the production of concrete: "All of these influencing factors are complex in themselves, but the complexity is vastly compounded by their intimate interrelation, and intensive research so far has not succeeded in disentangling any great number of these factors for individual scrutiny." This statement is to a lesser degree essentially as applicable to our state of knowledge regarding concrete as it was when written in 1948. This provocative statement, through implication, focuses attention on three important aspects of concrete research as conducted to date: first, it has of necessity been concerned primarily with the disentanglement of interrelated factors; second, it has been a slow, intensive, laborious, and therefore, costly procedure due to the complexity of the product; and third, progress toward the goal of making essentially "permanently durable" concrete has been relatively slight. Past concrete research reveals the definite probability that the degree of improvement believed possible of attainment can never be fully realized through improvement in the concrete, per se. Full recognition of this poses the question whether the established trend in concrete research should be reviewed with a view to achieving the desired goal more expeditiously and economically. Research has been and continues to be concerned solely with the improvement of the product itself; a review of the results obtained makes it increasingly obvious that the desired durability will never be attained because of the basic, unalterable, properties of the product which prevent adequate imperviousness necessary to "permanent durability." Such a review also indicates that improvements expected to accrue from pursuance of accepted trends in research will become proportionately insignificant compared to cost and effort expended.

In considering a fresh approach to the problem it will be of value to enumerate briefly the major recognized factors contributing directly or indirectly to concrete deterioration.

1. *Alkali-aggregate reactions*—Under certain conditions of temperature, moisture, and concrete proportions the minor alkalies of cement, sodium and potassium oxides, react with various forms of amorphous silica present in the aggregates to form reaction products commonly called silica gel, which through osmotic action creates hydrostatic pressures exceeding the tensile strength of the concrete.

2. *Freezing and thawing*—Water freezing and thawing in the surface voids, and in the interior capillary channels and voids, sets up stresses of sufficient magnitude to cause disruption and progressive disintegration of the concrete.

3. *Thermal incompatibility*—When two different aggregates, or an aggregate and mortar mix, are in intimate contact and have sufficient differential in either coefficient of linear expansion, or thermal diffusivity, or both, stresses set up when heat transfer occurs cause incipient fractures large enough to cause deterioration of the concrete when exposed to water and freezing action.

*"Symposium on Mineral Aggregates," *Special Technical Publication* No. 83, American Society for Testing Materials.

4. *Erosion*—Mechanical disintegration of concrete is caused by the abrasive action of water or wind-borne particles striking the surface.

5. *Cavitation*—Severe repeated stresses set up by cycles of negative and positive pressures on concrete surfaces in areas of turbulency stress the concrete beyond its elastic limit and cause progressive failure and disintegration.

6. *Sulfate attack*—Chemical attack occurs in which sulfates of magnesium, calcium, and sodium, in solution react with hydrated lime and hydrated calcium aluminate to form calcium sulfate and calcium sulfoaluminate, whose formation causes expansion and disruption of the cement paste.

TESTS WITH IMPERVIOUS COATINGS

The author in 1948 carried out limited research on the problem of concrete durability with a view toward establishing a new approach that would attain the envisioned goal more simply and, therefore, at less cost and more quickly. The aforementioned types of deterioration are generally recognized as the principal ones, but what may not be generally recognized is that the common denominator of all of them is water, in either liquid or crystalline form. We may then hypothetically conclude that concrete made adequately impervious to ingress water is essentially "permanently durable" concrete.

Believing that the basic, unalterable characteristics of concrete make it impossible to achieve effective imperviousness through improvement of the product itself, the author concluded that such a goal could be attained only through development of an impervious, durable, and adherent surface coating. In order to fully explore the effect of an impervious coating it was considered valuable to test specimens cast from concrete containing aggregates known to exhibit poor durability when subjected to rapid cycles of freezing and thawing.

Test specimens

A combination of fine and coarse aggregates was selected whose extremely low durability factor (DFE), derived from change in dynamic modulus of elasticity calculated from changes in fundamental transverse frequencies, was attributed primarily to thermal incompatability. Initially three beams, $3\frac{1}{2}$ x $4\frac{1}{2}$ x 16 in. were cast from the same batch of test concrete, cured under standard curing conditions, one for control and two for test. The two test beams, at the completion of the standard 14-day curing period for the freezing and thawing test (CRD-C114-55) designed to test the relative durability of aggregates, were removed from the curing room, surface dried, and immediately coated with a continuous film of neoprene latex rubber approximately 1/16 in. thick. One week was allowed for the latex to attain full rubber properties.

Freeze-thaw tests

Beams (including control specimen) were then subjected to the standard rapid freezing and thawing test in accordance with applicable provisions of Corps of Engineers' method CRD-C 114. As was anticipated, and following pattern previously established, the uncoated control beam deteriorated rapidly

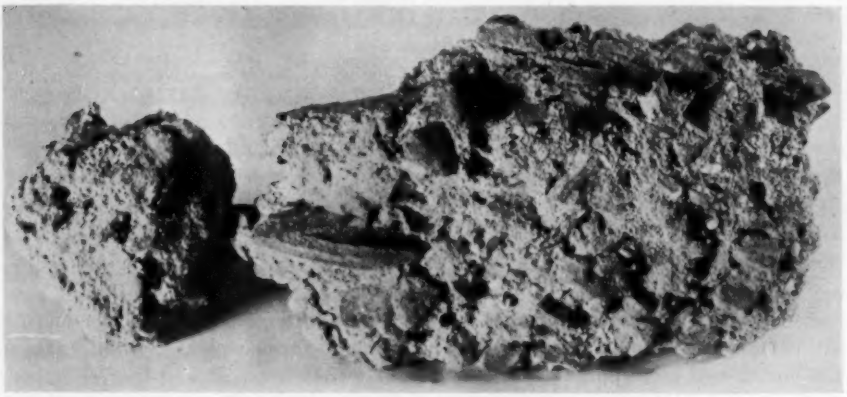

Fig. 1—Control (uncoated) beam after 300 cycles of freezing and thawing

showing only 50 percent relative modulus of elasticity at 150 cycles, or a DFE of 25, and complete disintegration at the end of 300 cycles, as shown in Fig. 1. The two coated beams continued at 100 percent relative modulus of elasticity for 500 cycles. At this point in the test the coating of one test beam developed a small pin hole; from the time that this occurred this beam started to fail rapidly, emphasizing that the concrete became vulnerable to the freezing and thawing action only after subjection to ingress water. The test beam whose coating had remained intact continued to show no decrease in dynamic modulus of elasticity up to 1200 cycles at which time the test was discontinued. After removal of this specimen from test, half of the coating was removed for examination of the concrete (Fig. 2). No signs of cracking or other distress were noted and the specimen appeared to be the same as the day it was placed under test, a fact substantiated by the test results shown in Fig. 3.

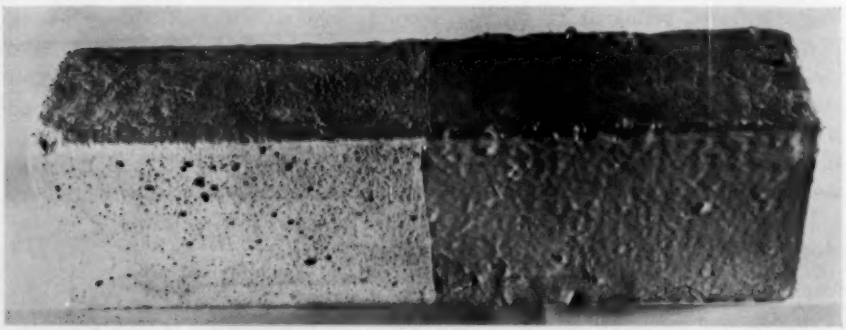

Fig. 2—Test beam after 1200 cycles of freezing and thawing. Heavier coating shown here than used on later test beams. Thickness of coating is immaterial in freeze-thaw tests so long as it remains impermeable

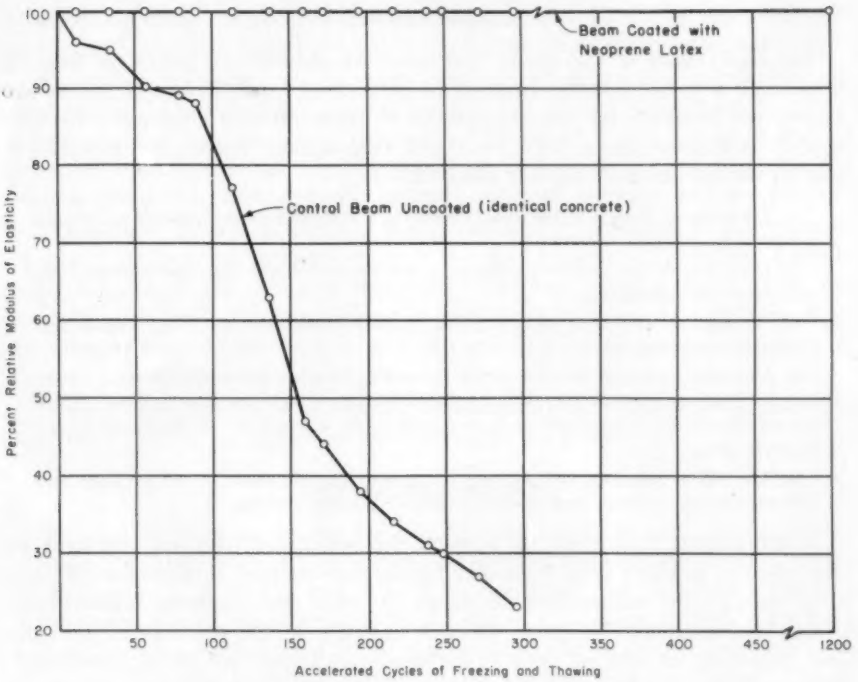

Fig. 3—Relative durability of coated and uncoated concrete

Abrasion tests

In addition to these tests the resistance of neoprene latex coating to abrasion by means of controlled sand-blast test was studied. In this test squares of identical concrete were sawed from a 1 in. thick slab; two were used for control and left uncoated, and the two test squares were coated with latex to a thick ness of approximately 1/16-in. In 1 hr under controlled sand blast it was found that the control specimen had eroded to a depth of approximately 7/8 in., while the coated specimens showed little or no distress to the coating after 5 hr of continuous test when examined under low power magnification.

While this experimentation was being carried out, the U. S. Bureau of Reclamation was conducting tests to determine the relative resistance to cavitation of various types of coatings. Results of these tests indicated that of the materials tested, neoprene latex exhibited the greatest resistance, bearing out a belief held by the author at the time, that resilient materials applied as coatings to concrete areas subjected to cavitation would be more effective than brittle materials such as manganese steel which had been employed at times.

CONCLUSIONS

The significance of this study and the work of others[1-7] lies in the greatly broadened scope of possible benefits to be derived from the use of effectively waterproof coatings for the attainment of more durable concrete. Certain hypothetical conclusions may be drawn that appear reasonably possible of substantiation through further research:

1. Exclusion of ingress water from concrete to a sufficient degree achieves essentially permanently durable concrete.
2. Durability of concrete thus achieved is not dependent upon the use of compatible or nonreactive aggregates.
3. Employment of proper coatings greatly increases resistance to deterioration caused by abrasion and cavitation.
4. Exclusion of ingress water from the interior and surface pores of concrete is necessary to make concrete invulnerable to attack by known deterioration factors. Water cannot reasonably be expected to be excluded solely through the improvement of the concrete, per se.
5. The logical approach to achieving the goal of durable concrete more rapidly and economically is to study and develop effective, durable coatings.

In advocating that concrete research be redirected from an endeavor to improve the product itself to achieving the desired goal through use of suitable coatings, the author does not mean to imply that neoprene latex rubber is the best coating. Neoprene latex is unstable in the presence of sunlight and, therefore, unsuited at least to portions of structures subjected to sunlight. However, neoprene latex may indeed prove a feasible coating for use on portions of structures not subjected to sunlight, especially areas subjected to erosion and cavitation, such as intake structures and sluiceways.

The author recognizes that the degree of saturation of concrete is a most important factor in its ability to withstand freezing and thawing as brought out by Powers.[1] Undoubtedly the degree of saturation of any concrete to be protected by a surface coating would have to be carefully checked to assure that it was below the critical point at the time the coating was to be applied. A related problem is determination of the degree to which internal moisture contained in the concrete would have to be reduced to effectively inhibit progressive alkali reaction when reactive aggregates are employed. These considerations, and many others, will be problems for further research to evaluate and solve, but their satisfactory resolution would not appear impossible.

RECOMMENDATIONS

The results obtained on limited test work, while not considered conclusive in all respects, definitely point the way toward a fresh approach to the goal of concrete research. It appears that durable concrete, economically produced, is more certain of realization through development of adequate surface protection than through slow and laborious improvement of the concrete itself. Concrete is essentially a porous product, and efforts to improve it to the point

of adequate imperviousness necessary for permanent durability seem destined to failure because of its inherent structural and mineralogical characteristics.

At present the engineer concerned with use of concrete as a construction medium has no alternative if local aggregates are inferior other than the costly importation of more durable materials; if he could choose on an economic basis between obtaining superior materials and providing an impermeable surface protection, design would be more rational, durable, and economical.

The author, therefore, strongly recommends that the focus of attention in concrete research be directed toward the development of suitable, durable surface coatings for concrete that will effectively exclude water from the concrete and present a high degree of resistance to abrasion and cavitation. It would appear, after review of several possibilities, that the coating medium may be one of the synthetic plastics.

It may well be that in the future the slogan, "Save the surface, and you save all," will be as applicable to concrete as to wood.

ACKNOWLEDGMENTS

The author is indebted to Bryant Mather, chief, Special Investigations Branch, Concrete Research Division, Waterways Experiment Station, Corps of Engineers, U. S. Army, and Albert Wiener, chief, North Atlantic Division Laboratory, Corps of Engineers, U. S. Army, for review of this paper and valuable assistance rendered in its preparation; and to Luther Farrior of the Concrete Research Division, Waterways Experiment Station, Corps of Engineers, U. S. Army, for his cooperative assistance in the research work.

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Discussion of this report should reach ACI headquarters in triplicate by Oct. 1, 1957, for publication in the March, 1958 JOURNAL.

Control Joints Regulate Effects of Volume Change in Concrete Masonry*

By G. A. MANSFIELD†, C. A. SIRRINE‡ and BENJAMIN WILK§

SYNOPSIS

Practical and easily applied methods of control jointing in use throughout Michigan virtually eliminate visible evidence of volume change in concrete masonry. If used wherever points of weakness are expected to develop noticeable cracking, these precautionary measures act to regulate crack location in a satisfactory and acceptable manner. Undesirable fractures in concrete masonry will occur rarely, if at all, when good construction practice, embodying an effective control joint procedure, is followed in conjunction with structurally adequate design and proper architectural details.

INTRODUCTION

Formerly, the eventual appearance of cracking was an anticipated corollary to concrete masonry construction. The cause for some of these expected effects could be traced to inadequate design; some were the result of failure to use top quality materials; some were caused by substandard workmanship; and some were due to natural volume change taking place in the completed work.

In recent years, emphasis given to the importance of properly designed footings and wall sections, along with substantial improvements made in concrete masonry manufacturing processes, have proved helpful in reducing the frequency of fractures in concrete masonry. Although valuable assistance has been rendered by these advances, laboratory investigations of concrete masonry have indicated volume change, an inherent characteristic of the material, to be the important factor remaining uncontrolled as far as incidence of cracking is concerned. This reaction of concrete masonry to moisture and thermal variations requires an economical and esthetically acceptable solution to the problem of control.

Drying of concrete masonry units at the point of manufacture and maintaining the dried condition to completion of construction has proved helpful in reducing crack frequency. However, drying processes alone have failed to totally eliminate cracking problems in Michigan. Due to climatic condi-

*Received by the Institute Nov. 23, 1956. Title No. 54-5 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 1, July 1957, *Proceedings* V. 54. Separate prints are available at 50 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Oct. 1, 1957. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

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tions encountered in this area, complete drying of units accounts for an approximated 50 percent of the total volume change due to both moisture and thermal extremes.

Complete removal of contained moisture from concrete masonry units by standard plant practice is uneconomical and usually detrimental to the quality of the product. In addition, it is ineffective, since it does not reduce the effects

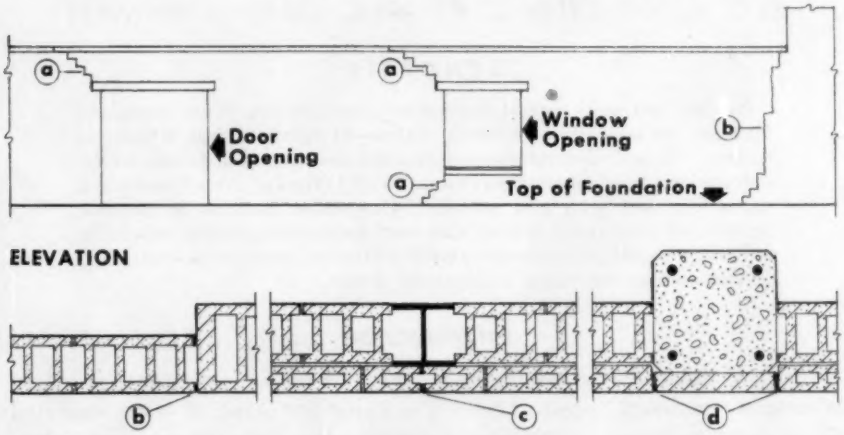

Fig. 1—Wall locations having a critical crack potential due to (a) partial stress relief provided by jamb sections of wall openings; (b) stress concentration at points of change in wall section height or thickness; unequal movement between embedded steel (c) or reinforced concrete structural members (d) and adjacent, unrelieved wall sections

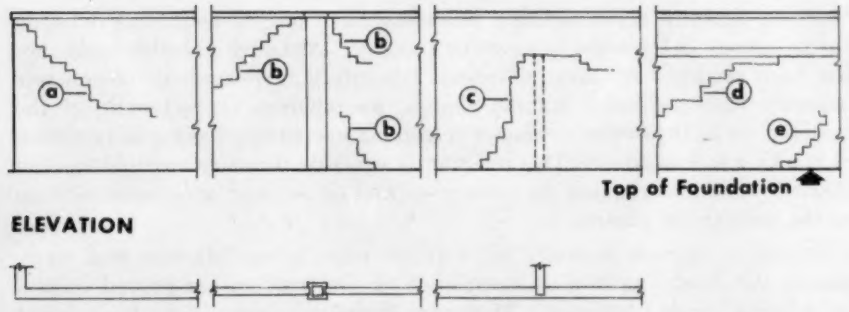

Fig. 2—Wall areas having a critical crack potential due to (a) resistive strength difference between bilateral corner construction and unrelieved, adjacent wall sections; (b) stress difference developed in unrelieved wall and pilaster construction; (c) stresses developed by masonry bonding partitions to unrelieved wall sections; unequal movement between unrelieved, hollow unit wall sections and restraining solid sections, such as bond beams (d) and foundation walls (e)

of volume change more than fifty percent below that of a completely saturated unit, nor does it resist the unavoidable effect of thermal differentials upon dimensional stability of the units. Consequently, further investigation to obtain a positive solution to the problem of crack control in masonry has been needed.

Conditions similar to those shown (Fig. 1-5) have been encountered sufficiently often to permit reference to them as typical of the problem. Points of weakness, stress concentrations, and restraining forces are all productive of cracking conditions in unrelieved wall sections undergoing any change in longitudinal dimension.

Such conditions usually become apparent soon after construction is completed, with most of the cracking observed to develop when wall sections are exposed to relatively dry, cold atmosphere on one face concurrently with exposure of the opposite face to warm, moist air. Incidence of cracks in a long, unbroken wall section of uniform height and thickness is undoubtedly a function of the quality of the concrete masonry units used, the type of aggregate selected for their manufacture, moisture content of the units when placed, the type of mortar used, and atmospheric conditions to which the completed construction is subsequently exposed.

Study of the problem included evaluation of the use of horizontal joint reinforcement, two-core units, and three-core units with thickened center face shells. Although offering some slight advantage, none of these methods offered a complete solution. Tests made to determine the effect of moisture content indicated some advantage in the use of concrete masonry units dried

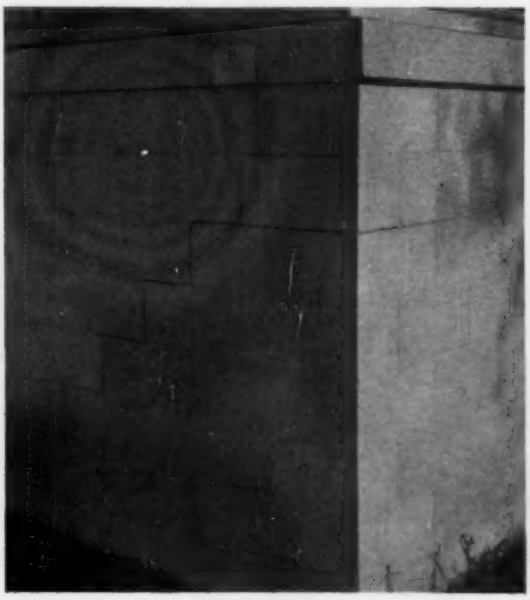

Fig. 3—Effect of volume change on an unrelieved wall section adjacent to bilateral corner construction

below the maximum of 40 percent of total absorption permitted by current ASTM specifications. No complete solution was disclosed by these tests.

Along with these investigations conducted by various organizations, the Portland Cement Association studied the use of relief joints to control the occurrence of cracking in concrete masonry. A vertical separation built into the wall section or a vertical saw cut in the face of the completed work was recommended, along with the use of a mastic material to replace mortar in the control joint. Both types of joints disclosed visible movement in use which obviously relieved stresses resulting from natural volume change.

About 1950, control joints were first used in a few Detroit area construction projects. Periodic and careful examination of results obtained by their use disclosed in every instance that these joints operated efficiently in relation to encountered atmospheric conditions. By reason of the total absence of cracks observed in panels controlled between properly located joints, coupled with visible indications of stress relief movement at the joints, it was conclusive that control joints could furnish adequate protection against the undesirable result of stresses commonly developed in concrete masonry construction. In no instance have cracks been observed to occur at points adjacent to properly located control joints or in areas where stress is relieved by the joints.

Fig. 4—Effect of volume change on an unrelieved wall section restrained by a pilaster and a sill bond beam

CONTROL MEASURES

Moisture control and reinforcement

As has been indicated, investigations have disclosed that something less than total crack elimination will result from the complete removal of moisture contained in the units or from reinforcement of critical stress concentration points by the use of two-core units, three-core units with thickened center face shells, and/or longitudinal reinforcement. However, an economical placement of reinforcing material in horizontal mortar joints does have an observed crack control value to the extent of distributing throughout a wall section the effects of stress concentration at a critical point. This means of reducing stress effect by dispersal becomes particularly important when combined with the opportunity for stress relief offered to a wall panel by its location between properly placed control joints.

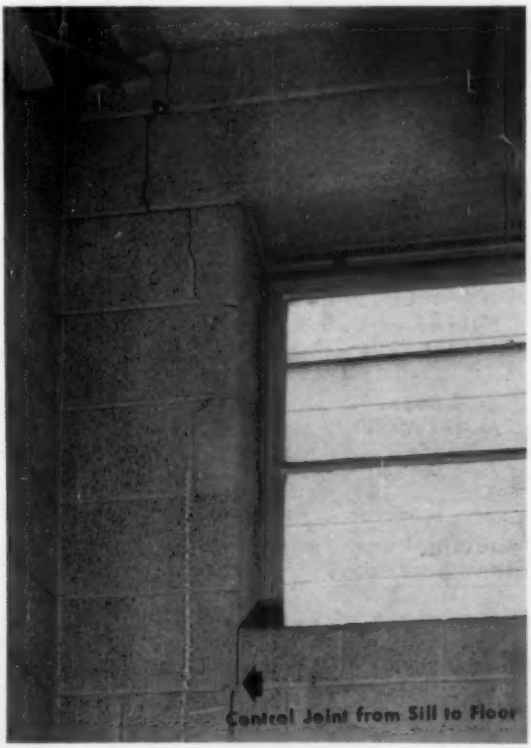

Fig. 5—Effect of volume change and load concentration on a point of weakness developed by stress relief at a window jamb and restraint exerted by an unrelieved, solid lintel

Although it is recognized that units containing moisture in excess of the 40 percent requirement previously cited are too frequently used in concrete masonry construction, enforcement of this moisture content limit required by the ASTM, and a further requirement of adequate protection against wetting of units before and during construction, is entirely possible and

practical. Both requirements are economically feasible and advisable to reduce volume change, which reaches an accelerated level when units contain moisture in excess of 40 percent of total absorption.

Control joints—location and construction

Control joints fabricated as shown (Fig. 6-12) are used in Michigan for all conditions encountered in concrete masonry wall construction. Experience in Michigan has proved that control joint spacing in walls of uniform height and thickness should not exceed 30 ft maximum, with an average spacing of approximately 25 ft indicated to be most effective in walls exposed to local climatic conditions. In walls of nonuniform section, control joints should be incorporated at points of maximum stress concentration, but within the limits stated above.

All openings in concrete masonry walls should be recognized as natural and desirable control joint locations. Although some adjustment in the control jointing pattern established for a wall may be required, the desirability of including vertical sides of wall openings as part of control joint construction makes such an adjustment mandatory in the development of a completely effective arrangement of control joints.

To avoid the occurrence of fractures due to a differential in movement between concrete masonry walls and structural framing members contained therein (Fig. 1), minimum clearances as shown in Fig. 9 must be provided in construction. Control joints shall be located on both sides of vertical framing members and spaced to coordinate with the control joint pattern established for adjoining walls.

Field observation has disclosed the necessity for corner construction to be separated from adjoining wall sections by control joints so located on each

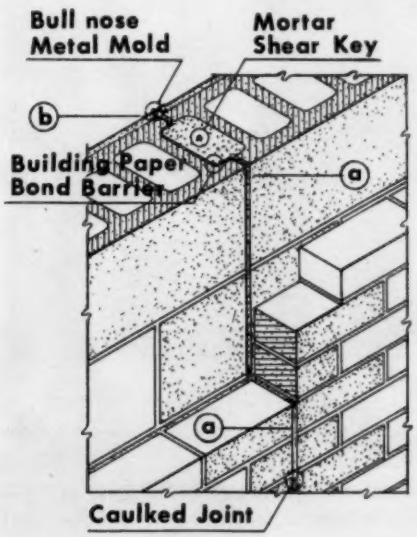

Fig. 6—Laterally stable control joint in concrete masonry showing (a) the vertical separation necessary in veneer material, and (b) the vertical separation required in plaster when applied directly to jointed concrete masonry walls

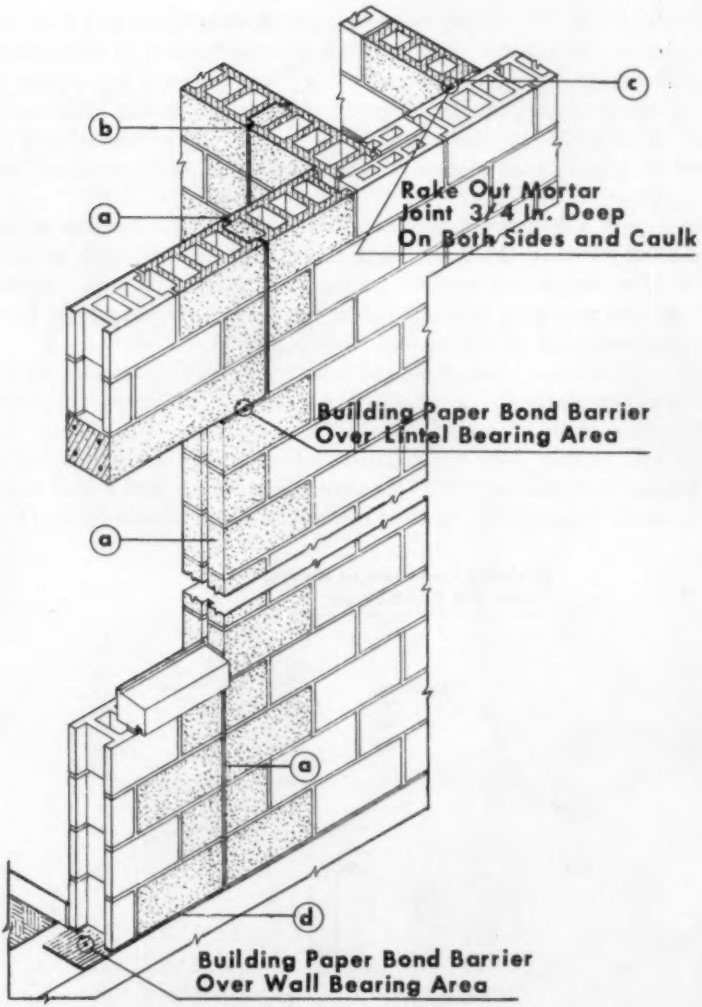

Fig. 7—Laterally stable control joint in concrete masonry at (a) wall opening; and (b) the juncture of an interposed, load bearing partition, masonry bonded. (c) shows a jointing procedure used for walls carrying no load, where lateral stabilization is not required. (d) indicates a method of relieving stress at the top of foundation walls

side of the corner that the total distance between them, measured around the corner, does not exceed that established for control joint spacing in the adjoining walls.

Although the use of pilaster units provides a natural control joint location on each side of the pilaster (Fig. 8), wall sections bonded to pilasters formed by cross bonding regular concrete masonry units should be control jointed in such manner as to provide adequate stress relief to the intervening wall sections. Individual circumstances will determine the advisability of including more than one cross-bonded pilaster in a wall panel contained between control joints.

In the case of a 50 percent true bond requirement, the location of laterally stable control joints in the interposing cross walls shall be such as to permit bond with the interposed wall to be made without difficulty. Interposing cross walls not requiring masonry bond for lateral stability may be joined to the interposed wall in the manner shown (Fig. 7c).

In wall construction where concrete masonry is used as backup for bonded facings of other materials, a vertical separation of all materials composing the wall section should be made at control joints (Fig. 6a). Plaster applied over lath and furring does not require vertical separation at control joints. Where plaster is applied directly to a concrete masonry wall a bull nose mold or flexible joint cover should be used in the plaster at control joints (Fig. 6b).

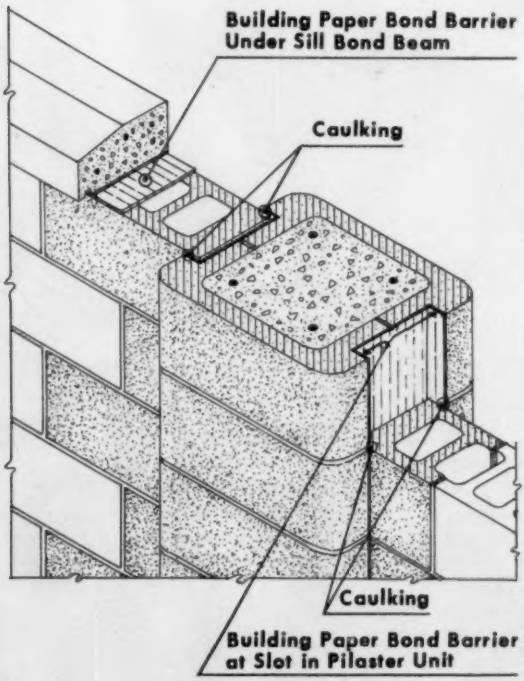

Fig. 8—Laterally stable control joint between pilaster units and adjoining wall sections, and a method of relieving stress developed between monolithic bond beam construction and concrete masonry units

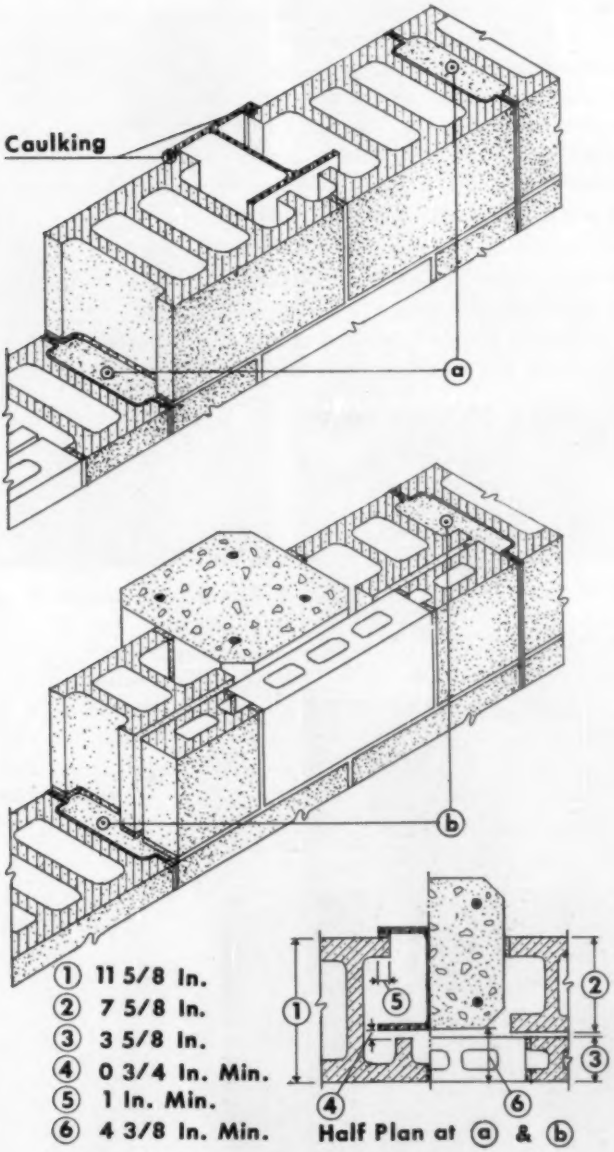

Fig. 9—Laterally stable control joint construction at (a) steel and (b) reinforced concrete structural members embedded in concrete masonry

All lintels over wall openings should have complete freedom of longitudinal movement over the bearing area beneath one end of the lintel. Where the vertical side of a wall opening forms part of a control joint, this slide plane should be located on the bearing area adjacent to that vertical side. Control joint construction continued above the lintel should be located at the head joint of the unrestrained end of the lintel (Fig. 7a). Where a control joint is formed in part by the vertical side of a window opening, joint construction should continue from the end of the sill adjacent to that vertical side vertically down to the top of the foundation (Fig. 7a).

To permit unrestrained movement at control joints, a longitudinal slip plane should be provided, wherever structurally practicable, between control-jointed wall sections and supporting foundations (Fig. 7d).

Wall reinforcement should be placed in horizontal mortar joints of masonry walls as an adjunct to control joints in the regulation of cracking in concrete

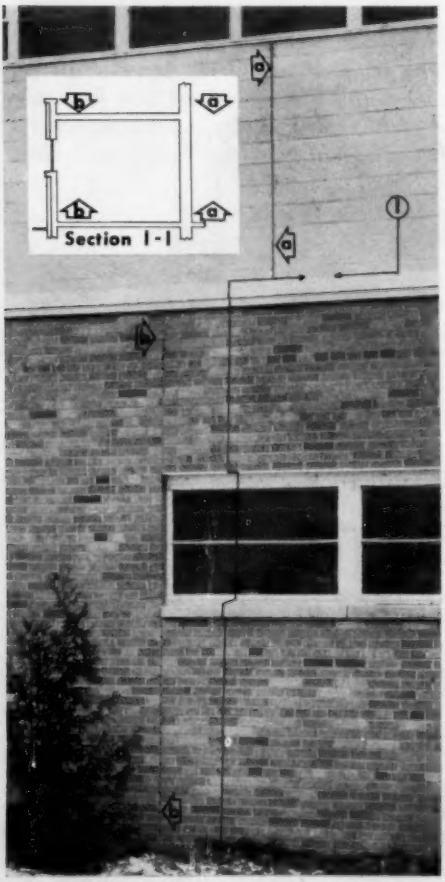

Fig. 10—Control joint construction in exposed concrete masonry (a), and brick veneer on concrete masonry (b)

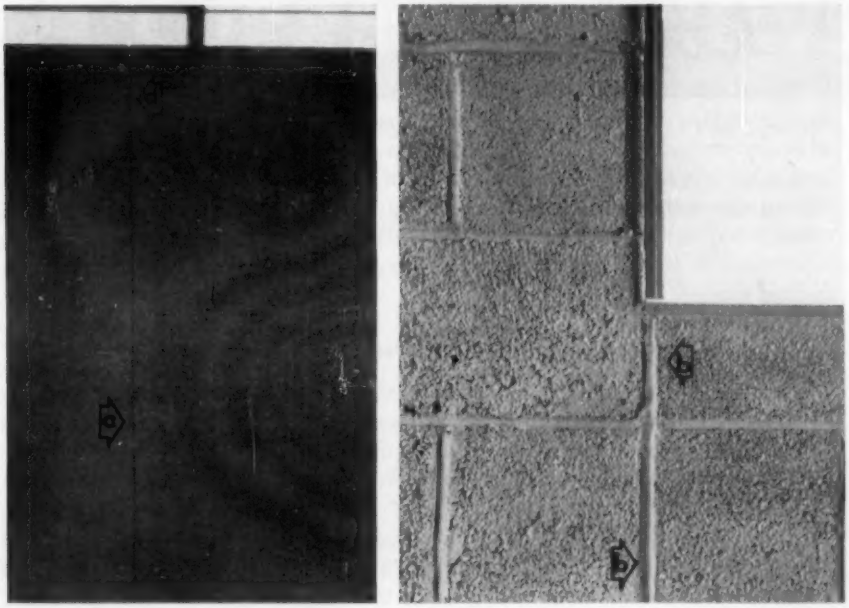

Fig. 11—Interior view of control joints similar to those shown in Fig. 10

masonry. This material provides an economical means by which wall panels between control joints are longitudinally tied into units. It is essential for horizontal wall reinforcement to be placed in properly control-jointed wall panels when the following conditions prevail:

a. Panels are subjected to differential exposure, such as a parapet wall continuing above an enclosure wall.

Fig. 12—Control joint application for stress relief at corner construction

b. Panels have stress concentration due to a reduction in total cross section by the occurrence of wall openings in the area between control joints.

c. Panels require more lateral stability than is provided by unreinforced construction.

Materials required to fabricate the laterally stable control joints predominantly used in Michigan (Fig. 6a) are those in common use wherever concrete masonry walls are built. The fact that no special units are required, and that a negligible minimum of on-site tailoring of all materials is necessary, indicates fabrication economy. Observed performance has proved that control joints constructed in the manner shown provide maximum resistance to transverse movement in addition to performing their normal function of relieving longitudinal stresses.

CONCLUSION

The value of control joints in successfully regulating effects of volume change in concrete masonry has been proved conclusively in Michigan practice by comparison with performance in similar buildings constructed with and without control joints. Intelligent use of this protective measure is desirable in all concrete masonry construction, whether single walls, cavity walls or backup, exterior or interior, above or below grade. Control joints should be incorporated as a necessary part of such construction wherever cracking is likely to occur as a result of developed stresses due to volume change.

Separate provisions for relief of volume-change stresses in cast-in-place slabs and bond beams shall coincide with the control joint pattern established for concrete masonry walls of which they are a part.

In addition to the need for adequate control measures in successful concrete masonry construction, close cooperation of the architect and structural engineer is important in assuring both a workable and an attractive arrangement of control joints, developed on the premise that workability is the first consideration. A continuing insistence on the part of the architect, engineer, and contractor to use only those materials which meet applicable requirements of approved specifications, and to adhere strictly to approved specification practices during construction, is of further and equal importance in the development of crack-free concrete masonry structures.

Discussion of this paper should reach ACI headquarters in triplicate by Oct. 1, 1957, for publication in the March 1958 JOURNAL.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Analysis of certain interconnected skew-bridge girders

ARNOLD W. HENDRY and LESLIE G. JAEGER, *Proceedings*, Institution of Civil Engineers (London), V. 6 (session 1956-57), Jan. 1957, pp. 79-106
Reviewed by ARON L. MIRSKY

Authors' method of grid-frame analysis by distribution of harmonics (*Proceedings*, ICE, Dec. 1955, pp. 939-971; "Current Reviews," ACI JOURNAL, Sept. 1956, p. 327; and others) is applied to the analysis of skewed bridges with three or four longitudinal girders of negligible torsional stiffness. Application of the results to bridges with more than four girders is outlined. The effect of varying the angle of skew is investigated. Numerical examples are given, and good agreement between experimentally-obtained and calculated (theoretical) values of load distribution is indicated.

Old bridge prestressed in place

Southwest Builder and Contractor, V. 128, No. 15 Oct. 12, 1956, pp. 36, 38
HIGHWAY RESEARCH ABSTRACTS Mar. 1957

Prestressing was specified for an existing bridge to raise its loading capacity to the H-20 requirements. The original bridge, of reinforced concrete T-beam construction, with a single span of 42 ft and having a loading capacity of H-15, was built more than 20 years ago. It was required that the existing bridge be widened by adding a new reinforced concrete T-beam span, to provide the over-all width of 205 ft 4 in. necessary to carry six lanes of freeway and four lanes of frontage road.

Pavement was removed at each end of the bridge to permit construction of anchorages and the cutting of holes in the end diaphragms of the abutments for the passage of cables. Steel channels were placed at midspan in the concrete diaphragm between girders, to serve as saddles for the prestressing cables. Because the bridge was built on a skew, it was necessary to construct offsetting triangular anchoring seats that would allow the anchors to take the stress at right angles.

Some cost data on prestressed concrete bridges

Bulletin No. 144, Highway Research Board, Apr. 1957, 34 pp., $0.80

Includes four papers dealing with comparative costs of prestressed concrete bridges and conventional construction in Massachusetts, Florida, California, New Jersey, New York, and Pennsylvania.

The particular projects discussed are illustrated both as to design, details, and construction.

Minimum transverse strength of slab bridges

P. B. MORICE, *Magazine of Concrete Research* (London), V. 8, No. 23, Aug. 1956, pp. 65-68
AUTHOR'S SUMMARY

A theoretical analysis of the modes of failure and ultimate strength of slab bridge shows that the normal method of elastic analysis leads to a design in which the transverse flexural strength is considerably greater than that required to give a factor of safety in the structure as a whole equal to that of longitudinal elements considered alone. It is suggested therefore, that in the design of

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slab bridges advantage can be taken of this by permitting moderate tensile stresses when calculating the degree of transverse prestressing required, and that in reinforced slab bridges the amount of transverse reinforcement can be appreciably reduced.

Construction

New look in platform roofs

Railway Age, V. 142, No. 6, Feb. 11, 1957, pp. 37-38
Reviewed by ARON L. MIRSKY

Reinforced concrete pillars, steel tube girders, and large prestressed concrete "tiles" combine to form modern railroad platform shelters at Winterthur-Grueze, Switzerland. Pillars are hollow, and contain equipment, vending machines, and other facilities and amenities.

Slipforms for apartment house construction (in Dutch)

H. E. WESTENBERG, *Cement* (Amsterdam), V. 8, No. 23-24, Dec. 1956, pp. 549-597
Reviewed by JOHN W. T. VAN ERP

Advantages of slipform construction are enumerated. In addition to reducing formwork and scaffolding, and shorter construction period, there is less exposure to cold weather—slipform scaffolds can incorporate additional weather protection. Economy is considerable as soon as more than eight floors are to be made. However, structures of as few as four floors have used slipform construction and proved economical. Precision of dimensioning is also mentioned. This method of construction is used so much in Norway, Sweden, Denmark, Finland, and Germany, because of the possibility of working at low temperatures, sometimes down to −4 F.

Acres of warehousing roofed with thin-shell, precast panels

Contractors and Engineers, V. 54, No. 4, Apr. 1957, pp. 44-47

A 35-acre facility for the U. S. Air Force at McClellan Air Force Base includes warehouses based on an improved standard design. Column and girder frames are cast in place; 33 x 5-ft precast thin-shell roof panels and tilt-up walls enclose the structure.

Building construction and fire protection

G. J. LANGDON-THOMAS, *The British Constructional Engineer*, V. 7, No. 4, Mar. 1957, pp. 33-35

Discusses the requirements and development of British standards for fire resistance of various building elements.

Budni tunnel—Its design and construction

K. SITARAMAN, *Journal*, Institution of Engineers (India), V. 37, No. 3, Part 1, Nov. 1956, pp. 159-202
AUTHOR'S SUMMARY

Peculiar and adverse ground conditions were met with due to the presence of several bands of soft clayey material in the cutting and serious criss-cross jointing in the rock formation. This necessitated the construction of an artificial full section tunnel inside the narrow cutting. The paper describes the special methods devised for designing the tunnel, taking into consideration passive pressures that would be mobilized by the sides of the cutting, and the several novel features adopted in the construction of the 820-ft tunnel, completed in 82 days.

Tests with light steel beams for masonry arch floor construction (Versuche mit Stahlleichtträgern für Massivdecken)

G. WEIL, *Bulletin* No. 119, Deutscher Ausschuss für Stahlbeton (Berlin), 1955, pp. 15-36
Reviewed by H. H. WERNER

Various types of German light-gage and open-web steel joists in combination with grouted-in filler block for the construction of floor slabs are investigated and tested. Practical aspects concerning stiffness, stability, mortar embedment, and corrosion resistance are explored and evaluated.

Dams

Avon Dam

Civil Engineering and Public Works Review (London), V. 51, No. 605, Nov. 1956, pp. 1237-1241

Mass concrete gravity dam under construction in England is 870 ft long and approximately 120 ft high. A pozzolanic "Trief" cement with low heat properties was used for the mass concrete. Precast concrete facing blocks were used on both upstream and

downstream faces. Provision for a future 15-ft extension of height was made by providing ducts for future vertical prestressing elements to be anchored in the rock foundation.

Description of repairs to spillway piers of Keokuk Dam

RALPH L. SHELTON and FRANK L. BURGRABBE, *Proceedings*, ASCE, V. 82, PO6, Dec. 1956, pp. 1133-1 to 1133-13

AUTHORS' SUMMARY

Describes the rehabilitation of the spillway piers of this dam after more than 40 years of service. Deteriorated concrete was removed and replaced with temperature-reinforced concrete anchored to sound concrete of piers.

Observed behavior of several Italian arch dams

DINO TONINI, *Proceedings*, ASCE, V. 82, PO6, Dec. 1956, pp. 1134-1 to 1134-26

AUTHOR'S SUMMARY

A summary of the results obtained from the analysis of deflection, strain, temperature, and other measurements on nine Italian arch dams. The important way in which temperature changes influence arch dam behavior is shown. The results of testing for foundation modulus by seismic methods before, during, and after construction of arch dams are described.

Design

Dynamic response in three dimensions of linear elastic structures to independent motions of multiple supports

RALPH E. BLAKE and GEORGE J. O'HARA, *Report* No. 4739, Naval Research Labortaory, May 15, 1956, 23 pp., $0.75 (available from Office of Technical Services, U. S. Department of Commerce, Washington, D. C.)

The equations of response of three-dimensional linear elastic structures, having rotational and translational inertia, to dynamic loading by foundation motions, are derived. The case of steady-state vibrations due to continuous harmonic foundation motions in arbitrary directions for all the foundations as well as one foundation is investigated, and the equations of response derived. The general equation for stress at any point is shown, and an example of its use is followed through.

Design of grid frame and flat slab bridges (in Dutch)

J. W. HOFFMAN and B. W. VAN DER VLUGT, *Cement* (Amsterdam), No. 19-20, Aug. 1956, pp. 479-486

Reviewed by JOHN W. T. VAN ERP

Described methods of design are suitable for concrete and prestressed concrete construction, but can also be used for concrete slab and steel-beam bridges. The design method, developed by Guyon and expanded by Massonet, is based on the anisotropic plate theory whereby it is possible to calculate the moments in the cross beams when the distribution of the loads over the main girders is given. The influence of torsional rigidity is also considered and although this greatly complicates the design work involved, it leads to very accurate results. Best results are obtained with an extensive grid work, that is, with a structure of a great number of girders and cross beams, but the method can also be used for a bridge with only a few main girders and only one cross beam. Numerous charts simplify the numerical work involved.

Continuous shell (Kontinuerlige skall)

OLAV OLSEN, Akademisk Trykningssentral Blindern, Oslo, 31 pp.

Reviewed by GUNHARD A. ORAVAS

This work investigates the curved boundary conditions of thin circular cylindrical shells. Analysis is carried out similar to Aas-Jakobsen's well known approximate procedure.* The fundamental differential equation is solved by a method originated by Odquist. The ensuing solution is applied to the following problems: cylindrical silo structure supported by six columns; continuous and fixed-end shells; analysis of stiffening ribs; and perturbation stresses brought about by stiffener diaphragms.

In addition comparisons of stress distribution are made for shells with simply supported and continuous circular boundary. Conversion factors are proposed to convert stresses in a simply supported shell into those of a corresponding continuous shell.

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*"Uber das Randstörungs-problem an Kreiszylinderschalen," *Bauingenieur*, No. 29-30, 1939.

R. Ohlig treats the problem of silo structures on columns identically in "Räumliche Tragwerke des Stahlbetonbehälterbaues," *Beton- und Stahlbetonbau* V. 48, No. 10, Oct. 1953, pp. 233-243.

An unusual staircase

Concrete and Constructional Engineering (London), V. 52, No. 4, Apr. 1957, pp. 141-143

Describes an unusual staircase designed with supports at the floors only. The flights and half-landings between floors were considered as V-shaped floors in a vertical plane.

Basic reinforced concrete design—Elastic and creep

GEORGE E. LARGE, Ronald Press Co., New York, 2nd Edition, 1957, 527 pp., $7

An extensive revision to the original edition bringing it completely up-to-date.

The book is eminently suitable for both textbook and reference purposes. The student is offered material in an easily understood form proceeding from simple illustrative examples to a complete development of basic theory.

The practicing engineer intending to use the textbook as a reference will appreciate its complete revision on the basis of the latest ACI Building Code including such new topics as design for shrinkage and creep, both for control of stresses and deflections; ultimate strength design; prestressed design; and an excellent chapter on flat slabs, including flat plates with various types of shear heads, both reinforcing and structural. Both students and practicing engineers will appreciate the generous use of diagrams and sketches to illustrate structural action in the theory and the wide selection of practical numerical examples to illustrate applications.

Earthquake stresses in building floors

CHARLES S. GLAZEBROOK, *Proceedings*, ASCE, V. 82, ST6, Nov. 1956, pp. 1098-1 to 1098-6

AUTHOR'S SUMMARY

A method to determine the shears in the floor panels and the direct loads in the beams of a floor system which are required to distribute horizontal loads caused by earth shock.

Method consists of determining the horizontal loads and their points of application which come on to the floor from above and the reactions and their points of application below the floor. The analysis is then made statically determinate by assuming that the floor beams in a bay of the building resist "overturning" moment in proportion to their distances from the center of gravity of the group of beams in the bay.

Internal ties in slope deflection and moment distribution

MORRIS OJALVO, *Proceedings*, ASCE, V. 82, ST6, Nov. 1956, pp. 1096-1 to 1096-17

AUTHOR'S SUMMARY

A procedure which can be used in the slope-deflection and moment distribution methods of analysis for joints of a rigid frame free to translate. Makes use of imaginary internal ties. In moment distribution these ties may be considered as the restraints which prevent joint translation during the balancing of the moments. The procedure becomes increasingly useful for the more complicated structures and does not complicate the analysis of the simpler structures. The versatility of moment distribution and slope-deflection is retained or enhanced.

Influence lines for circular ring redundants

HENRY M. LUMMIS III, *Proceedings*, ASCE, V. 82, ST6, Nov. 1956, pp. 1097-1 to 1097-22

AUTHOR'S SUMMARY

The circular ring is an indeterminate structure which requires the use of indeterminate structural analysis for its design. By using the influence lines for the redundants presented with this paper, the circular ring can be analyzed as a statically determinate structure.

Elementary theory of structures

CHU-KIA WANG and CLARENCE LEWIS ECKEL, McGraw-Hill Book Co., Inc., New York, 1957, 387 pp., $7.50

A textbook designed to present the essential principles for a first course in structural analysis.

The material included begins with algebraic and graphic methods of resolving four systems, simple beam analysis, simple truss analysis, influence diagrams, bridges, deflections, rigid frames, and indeterminate trusses.

The indeterminate analysis portion includes the slope-deflection method and the moment distribution method. Numerous completely developed examples are furnished, answers for which are supplied at the end of the book.

An excellent text for its purpose and also suitable for self-study in methods of indeterminate structural analysis by graduate engineers.

Membrane stresses in an elliptic paraboloid (in German)

E. Tungl, *Oesterreichisches Ingenieur-Archiv* (Vienna), V. 10, No. 2/3, 1956, pp. 308-314
Applied Mechanics Review Mar. 1957 (Neuber)

By means of the stress function method of Pucher, author gives a solution for the membrane stresses in elliptic paraboloid shells. The solution contains eight functions in form of not-final sums, but their most important values are calculated and tabulated.

Tables of functions for short cylindrical shells

R. B. McCalley, Jr., and R. G. Kelly, *Paper* 56-F-5, ASME Fall Meeting, Denver, Sept. 1956, 24 pp.
Applied Mechanics Reviews Mar. 1957 (Luke)

Presents tables to facilitate solution of title problems. Basic theory is well known (*Theory of Plates and Shells*, by S. Timoshenko). Four cases are considered: (1) one end of shell is fixed; other end is displaced a given distance, but slope is held as zero; (2) same as (1), but unfixed end is rotated through a given angle with deflection held zero; (3) both ends are fixed and applied internal pressure loading is uniform; (4) same as (3), but loading varies linearly with length. For each case, tables are given for deflection, bending moment, and shear as a function of nondimensional shell length and dimensionless shell parameter. Functions tabulated are elementary, as they are combinations of circular and hyperbolic functions.

Detailed design procedure for a prestressed, pre-tensioned, bonded concrete modified I-beam and composite girder

V. J. Brown, *Roads and Streets*, V. 100, No. 5, May 1957, pp. 115-125, 132

Presents a detailed solution to design problem typical of those which will confront many engineers as prestressed concrete construction increases under the nation's expanding highway program.

The problem as outlined is a design for submission as an alternate to a structural steel bridge. The design involves a composite bridge, precast, prestressed, modified I-beams composite with a cast-in-place slab.

Folded slab roof in glazed tracery walls for new church

Prefabrication (London), V. 4, No. 41, Mar. 1957, pp. 208-211

Illustrates design features, details, and forming for a steep, tent-like, folded plate roof covering a hall 50 x 60 ft. The folded plate is supported on rigid frames at each end and is prestressed by post-tensioning. Precast concrete grills on the side walls form a sash, being glazed directly.

Design and construction of engineering foundations

F. D. C. Henry, McGraw-Hill Book Co., Inc., New York, 1956, 547 pp. $9

Presents current practice and suggests possible future trends in design and construction of foundations in Great Britain and the United States. The author has assimilated much recent research literature, summarizing it, and illustrating its application to practical problems.

The book concentrates on applications of foundation design, developing these applications from fundamental principles of theory and research results.

The principal subjects included are: continuous, raft and individual footings; piers, piles, culverts, cofferdams, caissons, underpinning; design of formwork for concrete, bulkheads, and cofferdams. One hundred design problems are included in the appendix to illustrate applications. The principal value of the book will be as a reference for practicing structural engineers, but it may also be appropriate as a textbook, particularly for graduate courses in design.

The structure of high buildings

Ove N. Arup, *Transactions*, Institution of Civil Engineers of Ireland (Dublin), V. 82, 1955-56, pp. 23-50; discussions pp. 153-161

Presents general principles and application in recent practice for structural planning of buildings from 8 to 20 stories in height. Structural steel is dismissed from consideration for all buildings within this range as being more expensive than reinforced or prestressed concrete. Load bearing masonry walls are also dismissed. The paper discusses various schemes for framing roof and floor slabs, for designing the vertical load bearing members, and for designing the foundations.

Diagrams for calculation of elastically supported girders (in Danish)

JORGEN A. JENSEN, *Ingeniören* (Copenhagen), V. 66, No. 8, 1957, pp. 195-202

AUTHOR'S SUMMARY

The diagrams are based on the dimensionless coefficient $\epsilon = 6kEJ/l^3$ as parameter; where k = vertical displacement of support resulting from vertical unity force, E = factor of elasticity of girder material, J = moment of inertia of girder, and l = width of spans.

The diagrams give the influence lines for reaction R_n, bending moment at support M_n, bending moment at midspan $M_n^n + 1$, and end reaction R_o. The construction of influence lines for bending moment M_1 at first interior support, and midspan moment M_o^1 in first span, based on the above influence line R_o are shown. Diagrams are given showing the areas enclosed by the influence lines, also as functions of ϵ.

The diagrams give exact results only when k is the same for all supports, the girder homogeneous and the width of span constant throughout the girder. It is shown how the diagrams may also be applied in case of girders supported continuously on a homogeneous elastic subsoil.

The procedure of using the diagrams is shown by examples. The text on the diagrams is in English.

Recent shell structures (Beispiele neuerer Schalenbauten)

KARL BUYER, *Beton- und Stahlbetonbau* (Berlin), V. 52, No. 1, Jan. 1957, pp. 1-11

Reviewed by VALDIS LAPSINS

Design and construction features of various recently erected thin-shell roofed industrial buildings are analyzed. Differentiation is made between short and long shells, and identifying marks for stress distribution discussed along with application for design.

Structural design of Calder Hall nuclear plant

The British Constructional Engineer (London), V. 7, No. 4, Mar. 1957, p. 43

Summarizes a paper dealing with the design and construction of the nuclear plant at Calder Hall. Paper was presented at a symposium held by the British Nuclear Energy Conference.

Shearing strength of reinforced concrete slabs

N. S. SIH, *Proceedings*, ASCE, V. 83, ST 1, Jan. 1957, pp. 1149-1 to 1149-12

Evaluates tests by Richart, by Elstner and Hognestad, and proposes criteria for determination of the ultimate shearing strength of reinforced concrete slabs and footings subjected to concentrated loads based upon these tests. Two criteria are proposed and compared to the methods of the present ACI Code. The present Code is based on a criterion such that the shearing strength is practically independent of the actual total reinforcement stress (which occurs when a slab has a high percentage of reinforcement), and the second criterion for other conditions is based upon the reinforcement reaching its yield point stress.

Ultimate strength design under 1956 Building Code

RAYMOND C. REESE, *Proceedings*, ASCE, V. 82, ST6, Nov. 1956, pp. 1099-1 to 1099-8

AUTHOR'S SUMMARY

Shows how ultimate strength design is recognized for the first time in "Building Code Requirements for Reinforced Concrete (ACI 318-56);" explains that this is ultimate strength design, not limit design (moments being obtained by elastic frame analysis); shows the need of controlling shallow sections by deflection calculations; and points out that ultimate strength procedures are easier to apply and give more realistic results than working stress formulas.

Wind forces in multi-story frame design (Die Berechnung von Stockwerkrahmen fuer Windlast)

GEORG EHLERS, *Beton- und Stahlbetonbau* (Berlin), V. 52, No. 1, Jan. 1957, pp. 11-16

Reviewed by VALDIS LAPSINS

An attempt is made to simplify lengthy theories and to improve approximate methods in use for determining wind stresses in multistory buildings. Method of analysis is based upon the assumptions made. Author proposes investigation of wind stresses in frame structures by applying his own frame formula and making basic assumptions that there is no deformation of members and that the loads are applied at the corners of the frame.

CRSI design handbook

R. C. Reese, Concrete Reinforcing Steel Institute, 2nd Edition, 1957, 447 pp., $6

The original handbook has been brought up-to-date now including a short standard specification for concrete and reinforced concrete, tables for stirrups and web reinforcement based upon the 1956 ACI Building Code, standard flat slabs, and the "waffle" two-way joist flat slab.

As in the earlier version, this revision presents finished designs of reinforced concrete members giving concrete sizes and required reinforcement. Safe load tables eliminating the need for charts and diagrams present the designer immediately with concrete outlines and reinforcing steel required for a given load and span. Fundamental theories, some diagrams and charts have been included for those who prefer to make their own design computations. A brief summary of formulas for ultimate strength design is included with necessary charts. The designs given are particularly good for preliminary estimating for establishing sizes and clearances and for comparing different types of construction. Although mathematically correct for the limited conditions stated at the head of each section, the handbook is not intended to replace professional judgment of an experienced structural engineer necessary to obtain satisfactory and economical finished structures.

Effects of shrinkage and creep on cracked reinforced concrete beams (in Hungarian)

L. Kollar, *Mélyé pitéstudományi Szemle*, V. 5, No. 11, 1955, pp. 499-505

Hungarian Technical Abstracts V. 8, No. 3, 1956

Effects of shrinkage and creep on stresses and strains of cracked rectangular and T-section reinforced concrete girders are investigated. The linearity of creep, affinity of the curves of shrinkage and creep, and constancy of modulus of elasticity are assumed for the analysis. After describing the accurate method of calculation a more convenient procedure is published. The conclusion was drawn that while changes in stress distribution due to shrinkage and creep are of no importance, the effects on deformation are significant.

New methods in structural analysis (Nuevos métodos en calculo estructural)

G. Yassinsky, Tipografia Italiana, A.C., Caracas, Venezuela, 1955, 143 pp.

Reviewed by Emilio Rosenblueth

The book is divided in two parts. The first deals with a method with several variations for solving continuous rectangular frames by a systematic solution of simultaneous equations by substitution. In the case of continuous beams and simple frames the method is straightforward and just as quick as the step-by-step procedure. It has the advantage over the latter procedure of not requiring differences between large quantities. But its efficient application requires use of several charts found in this work. The chief objection this reviewer finds against methods of this type is that the necessary operations bear no immediate relation to physical concepts, so the analyst easily loses grasp of the physical problem. In more complicated frames the author makes certain assumptions as to the rotations at the far ends of columns, which allow him to arrive quickly at a first approximation. The result can be improved by successive approximations, but the first or second approximation is usually satisfactory.

The second part concerns several design charts for reinforced concrete beams and columns. Apparently the author assumes conventional linear theory with permissible stresses independent of loading conditions. Charts on tracing paper facilitate adapting copies to specific conditions.

This work also includes a number of random thoughts and shortcuts in structural analysis and design.

Expected crack formation in flexurally loaded beams (in Dutch)

E. A. F. Huber, *Cement* (Amsterdam), V. 8, No. 23-24, Dec. 1956, pp. 587-591

Reviewed by John W. T. Van Erp

By means of charts, calculation of crack width is shown for conventional design methods with various values of n and also for ultimate strength methods of design. Designs are based on the Netherlands concrete code. Maximum permissible width of cracks is established first and from this the required amount of steel reinforcement is derived.

A contribution to expedient shaping of calotte shells over rectangular base (Ein Beitrag zur zweckmässigen Formgebung der Kappenschalen über rechteckigem Grundriss)

P. Csonka, *Publications*, International Association for Bridge and Structural Engineering, Zurich, V. 16, 1956, pp. 71-84

Reviewed by Gunhard A. Oravas

In this paper author applies his methods of analysis to translational shells (See "Current Reviews" ACI Journal, Apr. 1956, *Proc.* V. 52, p. 897, and Dec. 1956, *Proc.* V. 53, p. 612) formed by cycloidal generators, diagonal translational shells, compound calotte shells and paraboloidal translational shells described by a parabolic generator with a decreasing curvature towards the apex of the shell.

All derived solutions exhibit the same striking simplicity which is characteristic of all Dr. Csonka's contributions in the theory of thin shells.

Influence areas for beam grids (Einflussflaechen fuer Kreuzwerke)

H. Homberg and J. Weinmeister, Springer-Verlag, Berlin, 2nd Edition, 1956, 156 pp., 43.50 DM

Reviewed by Fritz Kramrisch

The book contains a valuable collection of tables for the structural analysis of beam grids; although the book is kept as general as possible, it seems to have been prepared primarily as an aid for the design of bridge floors.

About one-quarter of the book is devoted to theoretical explanations of the formulas and practical examples. In spite of that much text, the content of the book seems to be reserved only for engineers who are thoroughly familiar with this subject, and can hardly be recommended for quick reference because the handling alone of the notations for safe application of the tables requires considerable study. A clearer presentation of the notations and a better cross reference between examples and tables would have permitted a wider distribution and greater use.

The tables have been calculated under full consideration of the bending stiffness of the load-distributing cross beams; in addition the torsional stiffness of the girders has been included approximately and the shearing resistance of the slab itself is discussed and approximate methods are recommended to consider this influence also.

The tables give the values of the ordinates and areas of the influence lines for the main girders and cross beams of grids consisting of several parallel main girders with framed-in cross beams of different numbers.

The following conditions are treated: girders, supporting different numbers of cross beams, over one opening, either simply supported at both ends or fully restrained at one or at both ends; girders, supporting different numbers of cross beams, being continuous over two or three bays having different span ratios.

Where the number of the parallel acting girders and beams becomes relatively great a two-dimensional system of an orthotropic slab has been substituted which provides a close approximation and simplifies the calculation. The book also furnishes influence values for such strips of orthotropic slabs, either simply supported or fully restrained along both longitudinal edges. There are also tables giving the reactions of beams having three to ten, and infinite many, elastic supports.

Analysis of vibrating systems which embody beams in flexure

R. E. D. Bishop, *Proceedings*, nstitution of Mechanical Engineers (London), V. 169, No. 51, 1955, pp. 1031-1050 (including discussion)

Reviewed by Aron L. Mirsky

Theory and application of receptance (Duncan's) method of analyzing free and forced motion of uniform beams, such as occur in frames and bents. Tables of receptance functions and their numerical values are given for ease of computation.

Materials

Tests of uniformity of mortar strengths of cement samples from same source

Stanton Walker and Delmar L. Bloem, *Publication* No. 71, National Ready Mixed Concrete Association, Washington, D. C., Jan. 1957, 24 pp.

Significant shipment-to-shipment variations in strength were found for five "outside" sources of cement. Comparisons with the variability of a "control" sample proved conclusively that the shipment-to-shipment strength differences were attributable to changes in the properties of the cements and not to lack of uniformity in testing.

Expanding cement (Cement rozszerzajacy sie)

I. Ahrends and B. Dybowska, *Cement-Wapno-Gips* (Stalinogród), No. 4, 1955, pp. 82-86
Polish Technical Abstracts No. 1 (21), 1956

There has arisen a demand for special cements causing no shrinkage in mortars, and for cements which expand slightly in the initial setting period. Additions of expanding mixtures to ordinary cement and tests of a semi-technical production of the kinds of cement mentioned were experimented with. The mixture added consisted of two components: burnt gypsum, and hydrated tetracalcium aluminate ($4CaO \cdot Al_2O_3 \cdot 13H_2O$) obtained by treating slag of high aluminum content with an aqueous solution of $Ca(OH)_2$. The two components of the mixture interreact when the cement is admixed with water, and form a hydrated calcium-sulfoaluminate ($3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 0H_2O$), which has an expanding effect on the cement "dough." Pilot tests demonstrated that ordinary cement admixed with 25-30 percent of an expanding mixture can yield a good, slightly expanding, and watertight cement.

Sintering and lightweight aggregates

Adolph F. Leitner, *Pit and Quarry*, V. 48, No. 8, Feb. 1956, pp. 94-96, 105; No. 9, Mar. 1956, pp. 104-106, 110

First article discusses the nature and principles of the sintering process. Second part treats some of the practical considerations in planning a commercial sintered lightweight aggregate manufacturing operation.

On the testing of cement

Cembureau, Malmö, Sweden, 1956, 57 pp., $1.40

A report of Cembureau's committee on cement standards including methods of testing and strength of cement. The object of the committee work originally was to develop a uniform export specification for cement. A more immediate objective is to promote adoption of proposed uniform test methods as national specifications in as many countries as possible. This report is concerned mainly with definitions, chemical composition, soundness, setting time, and mechanical strength of cement.

Review of the portland cement standards of the world—1955

Cembureau, The Cement Statistical and Technical Association, Malmö, Sweden, 1956, 93 pp., 18 shillings

A compilation of standard specifications for cement valid in different countries. Presented by countries in alphabetical order, information is given in both text and tables. When possible, official specifications were used as the basis for the tables; the absence of details in some cases means that only incomplete excerpts were available.

Principal feature is the chapter entitled "Summary of Portland Cement Standards," which gives the reader the chief substance of the national cement specifications in 40 countries. General information on the chemical, physical, and strength requirements, and on various points relating to the testing of cement, is found in the chapter called "Portland Cement Standard Specifications and Test Methods," so worded as to be intelligible to all, even if they are not cement technicians. Finally an index listing available data on the application of standard specifications for cement all over the world, and comprising more than 100 countries. It indicates which countries have their own cement production, and whether they apply their own, or some other country's cement standards.

Lightweight calcium silicate hydrate: some mix and strength characteristics

W. H. Taylor and D. R. Moorehead, *Magazine of Concrete Research* (London), V. 8, No. 24, Nov. 1956, pp. 145-150
Authors' Summary

Presents the results of a study of some physical characteristics of autoclaved products, relating to mix proportioning and compressive strength, using five grades of silica flour that vary widely in specific surface area.

A limited number of results, obtained under an arbitrary set of conditions without batch replication, show approximately linear relations between (a) the unit weight of dry products and the water/solids ratio of mixes, (b) the ratio of the compressive strength to the unit weight of dry products and the water/solids ratio of mixes, and (c) the lime/silica ratio (for maximum compressive strength) and the specific surface area of the silica flour used.

Allowable stress in high-strength reinforcing steel (in Dutch)

E. J. VAN KOOT, *Cement* (Amsterdam), V. 8, No. 21-22, Oct. 1956, pp. 499-504
Reviewed by JOHN W. T. VAN ERP

Article demonstrates that these stresses are determined by the permissible crack width. According to widely accepted standards these crack widths are: for outside construction or construction in an aggressive atmosphere, 0.008 in.; for inside construction or in a nonaggressive atmosphere, 0.012 in. Allowable stresses are, in the first case, 28,000 psi and in the second, 31,000 psi. An even distribution of the reinforcement over the entire tensile area is important and maximum percentage of reinforcement is determined. High-strength steel can be quite satisfactory and economic as well, provided the above criteria are being followed.

Crystallographic data for the calcium silicates

L. HELLER and H. F. W. TAYLOR, Building Research Station, Department of Scientific and Industrial Research (England), 1956, 80 pp., 10s 6d

Most of the known calcium silicates, whether artificial or naturally-occurring, were re-examined and their properties have been described in a series of published papers.

In the present publication the optical and x-ray data have been collected and arranged in a form suitable for reference. The opportunity has been taken to include some observations not previously published.

Air entrainment in cement mortar

S. BALASUBRAHMANYAM, M. R. VINAYAKA and H. K. PUJARI, *The Indian Concrete Journal* (Bombay), V. 30, No. 5, May 1956, pp. 154-158
HIGHWAY RESEARCH ABSTRACTS Sept. 1956

The grading of sand has a considerable effect on the mortar made with it; for mortar used in masonry work, it is necessary to insure that the sand is well-graded to make the mortar adequately workable. Since air entrainment in concrete is known to reduce the adverse effects due to poor grading of the sand used, it is logical to expect similar benefits in mortar also. The desirable effects should be more pronounced in the case of crushed fine aggregate. Therefore, investigations were carried out to determine the effect of air entrainment on the properties of cement mortar made with river sand (natural sand) as well as with crushed aggregate dust (crushed fines).

From the review of the test data, the following conclusions are indicated:

1. Air entrainment in mortar leads to increased cohesiveness and plasticity. This is specially so in the case of lean mortars and crushed fine mortars which will otherwise be harsh.

2. Normally, entrainment of 5 to 8 percent air is enough to give desirable workability. A larger quantity of admixture is required in the case of richer mixtures than in the case of leaner mixes to entrain the same amount of air; also, crushed fines mortar requires more agent than natural sand mortar, assuming that the difference in the grading of materials tested does not have any appreciable effect on the entrainment of air.

3. As should be expected, air entrainment in mortar leads to reduction in unit weight.

4. As in concrete, air entrainment reduces the compressive strength of mortar; the reduction for each percent of air entrained is more in the richer mixes than in the leaner ones. Comparatively, the loss in strength due to entraining air is more in the case of fines mortar than in natural sand mortar.

5. Air entrainment does not adversely affect the permeability of the mortar.

6. A reduction in the linear expansion is noticed from the entrainment of air. Because of the increased workability resulting from air entrainment, the water-cement ratio can be reduced, the reduction possibly offsetting any shrinkage as well as other ill effects.

Pavements

Placing concrete reinforcement

Roads and Road Construction (London), V. 34, No. 400, Apr. 1956, p. 126
HIGHWAY RESEARCH ABSTRACTS Jan. 1957

Describes a simple machine called a "fabric layer" used in paving. The machine consists of a series of prongs or skids which hold up the fabric reinforcement in the concrete, allowing the correct amount of cover. It is designed to run along the rails provided on the site for the mechanical concrete spreading and compacting plant, and being attached to the concrete spreader by two tubular arms is propelled forward by this unit, the prongs,

or skids, thus being withdrawn at the same time as the concrete is laid. The "fabric layer" enables the full depth of concrete to be laid in one operation, obviating the necessity for the use of stirrups or spacers.

Sector analysis for concrete pavement load stresses

BENGT F. FRIBERG, *Proceedings*, ASCE, V. 83, HW 1, Jan. 1957, pp. 1153-1 to 1153-36
AUTHOR'S SUMMARY

This paper is an outline of sector analysis, an approximation method of elastic analysis for slabs subjected to tire-imprint loads and resting on yielding subgrades. The method is used for a wide range of loaded-area dimensions, and for slabs of limited dimensions, such as highway and airport pavements.

The principles and procedures are described, and sector analysis is applied to stress and deflection determination in slabs of conventional highway dimensions subjected to single-wheel loadings for which comparative test observations are available. Stresses computed by sector analysis are in line with test observations. Application to common highway axle loadings is illustrated and compared with test road observations.

Precast Concrete

Design of precast structural elements for multistory dwelling houses (in Hungarian)

L. BRETAN, *Magyar Épitöipar*, V. 4, No. 10, 1955, pp. 437-449
HUNGARIAN TECHNICAL ABSTRACTS V. 8, No. 3, 1956

Four types of structural design are treated. The first provides frames consisting of solid reinforced concrete columns, of girders having combined cross sections, and of ribbed panels of the trough type. According to the second, columns with box-like cross section, non-sagging girders and ribbed or plain panels are used. The third design utilizes split-end, solid reinforced concrete columns, cantilevered and hinged girders of combined cross section, and grooved-ribbed floor panels. The fourth type comprises box columns (with intermediate wall), load-bearing ribbed floor slabs, and flat lightweight floor panels spanning the space between the former.

Single-story industrial buildings in Bulgaria

I. DOGANOFF, *Prefabrication* (London), V. 4, No. 40, Feb. 1957, pp. 160-164

Precast doubly-curved shell roofs have been used for one-story factory buildings. The doubly-curved shells are shown in a variety of styles on a variety of building projects. In general they are approximately 1½ in. thick and provide clear bays of perhaps 18 x 50 ft.

Behold the lowly concrete block—It isn't lowly any more

House & Home, V. 9, No. 3, Mar. 1956, pp. 142-155

A well-illustrated presentation of architectural effects achieved by use of concrete masonry, using various patterns in laying up walls and by casting special patterns into the block. Several outstanding residential examples are shown.

Prestressed Concrete

Prestressing systems (Voorspansystemen)

H. C. STUART, *Cement* (Amsterdam), V. 8, No. 15-16, Apr. 1956, pp. 393-397
Reviewed by JOHN W. T. VAN ERP

One of a series describing various systems. Items covered are:

Leoba cable: looped at the stressing end, anchored by bond at other end. Strength of cable 5-8 percent smaller at loop end than at normal cross section is no objection because heaviest loading of cable is near middle at service and ultimate loads. No projecting anchorages beyond concrete. Length up to about 150 ft.

Leoba bars: 1-in. round bars suitable for shorter lengths, like lateral prestressing in slabs and bridges.

Baur-Leonhardt stressing block system: cables are looped around large diameter (6 ft) concrete block at stressing end. Advantage is uniform stressing of all steel and as steel is bundled together in one sheath, concrete is not broken up but left in large cross sections; large aggregate size can be used, low water-cement ratio, and concrete with little creep is obtained.

Prestressed concrete runways: History, practice, and theory

ALAN JAMES HARRIS, *Proceedings*, Institution of Civil Engineers (London), V. 6 (session 1956-57), Jan. 1957, pp. 45-66 (discussion, pp. 66-78)
Reviewed by ARON L. MIRSKY

An excellent review and critique. Author concludes, among other things, that full-scale prestressed runways have been built competitive in price with classic pavements, and that the high load-bearing capacity of such runways can be put to good use in several ways (reduction in expense of subgrade compaction, decrease in slab thickness, and/or increase in wheel loads and tire pressures).

Ferryboat pier in the port of Amsterdam (in Dutch)

M. KROES and P. GROENHART, *Cement* (Amsterdam), V. 8, No. 23-24, Dec. 1956, pp. 563-570
Reviewed by JOHN W. T. VAN ERP

The structure is a grid of prestressed prefabricated beams on concrete piles. The floor, total length of about 900 ft, consists of prefabricated slabs filling the grid openings and a topping of 6-in. concrete roadway with a 2-in. asphaltic cover. Expansion joints are 205 ft on centers. The longitudinal beams are of T-section 24 in. high. Erection was as statically determinate simple span prestressed beam. Continuity was obtained by installation of negative moment prestressing steel. Cross beams are also prestressed so that the entire deck structure between expansion joints is a prestressed statically indeterminate grid of beams. Concrete topping was vacuum concrete. Unusual is a basement structure hung between the concrete piles of the pier.

Viaducts of prestressed concrete (in Dutch)

J. C. MEISCHKE and A. M. SOELS, *Cement* (Amsterdam), V. 8, No. 23-24, Dec. 1956, pp. 576-585
Reviewed by JOHN W. T. VAN ERP

A large number of viaducts was constructed of prestressed prefabricated beams. The small viaducts of 30- to 36-ft span use pre-tensioned reinforced beams of inverted T-sections with cast-in-place fill. Larger viaducts have post-tensioned solid bar reinforcement of the Finsterwalder system (threaded anchorages, threads rolled, not cut). Prestressing steel tubes were fixed in the mild steel reinforcing cage by welding. Concrete ingredients were carefully batched to be sure of the 6400-psi, 28-day strength. Concrete quality was checked with concrete test hammer readings and proved in good agreement with the test cube compression tests. Another feature of the projects was the highly mechanized fabrication and erection of the beams, whereby, a minimum amount of skilled labor was needed.

On-site prestressing plant cuts concrete building cost

Engineering News-Record, V. 158, No. 16, Apr. 18, 1957, pp. 58-61

A temporary prestressing plant was used to fabricate concrete roof panels, girders, and columns for a 100,000-sq ft Los Angeles warehouse.

Concrete slabs

H. J. HOPKINS and W. M. SLATER, *New Zealand Engineering* (Wellington), V. 10, No. 12, Dec. 15, 1955, pp. 448-453
HIGHWAY RESEARCH ABSTRACTS Nov. 1956

In many bridges and other slab structures with length greater than breadth, the amount of transverse prestress required to give full distribution should be known accurately if the full economy of this type of construction is to be effective.

The authors investigate the behavior of units cast side by side and integrated without grouting with varying amounts of transverse prestress. The load-distributing properties were compared with those of monolithic slabs.

It is not possible at present to state whether the minimum transverse prestress is a function of the stress or whether the spacing of wires is a function of the slab thickness, breadth, or span, but it is suggested that it would depend on all four, giving the following design criteria: stress not less than 200 psi; spacing not greater than the least of $6D$, $L/4$, or $b/2$.

Cable friction in post-tensioning

T. Y. LIN, *Proceedings*, ASCE, V. 82, ST6, Nov. 1956, pp. 1107-1 to 1107-13
AUTHOR'S SUMMARY

The sources of frictional loss of prestress are discussed. Data for the coefficient of friction under various conditions are presented. Methods for measuring and for reducing frictional losses are described.

Kwinana jetty

PETER MURRAY and DEREK N. COLLETT, *Proceedings*, Institution of Civil Engineers (London), Part III, V. 5, No. 3, Dec. 1956, pp. 785-823 (including discussion)

Reviewed by ARON L. MIRSKY

To save time, precast concrete construction was selected for large jetty serving an oil refinery in Western Australia. Post-tensioned prestressed connections (joints) required precision casting of members, development of a special small lightweight prestressing jack, and use of a high-strength concrete caulking mix, mechanically rammed into the joint.

Some studies on the deflection of prestressed concrete beams

C. A. STRAUBEL and A. M. OZELL, *Journal of Prestressed Concrete Institute*, V. 1, No. 4, Mar. 1957, pp. 5-26

Discusses suitable values for modulus of elasticity of concrete to be used for the prediction of deflection in flexural members over a wide range of concrete strengths including extremely high concrete strengths. Suggests that an effective modulus of elasticity be used whose value is dependent upon the stresses, both tensile and compressive, in the concrete member. Suggests that the stress-strain curve for concrete is much better represented by a second degree parabola than the customary assumption of a straight-line relationship. Includes examples showing application of the effective modulus of elasticity method to predict deflections.

Properties of Concrete

A theory of the deformation and the failure of concrete

ERLING REINIUS, *Magazine of Concrete Research* (London), V. 8, No. 24, Nov. 1956, pp. 157-160

The theory developed attempts to explain such characteristics of concrete as the mechanism of cracking and failure, deformations at failure, variations of Poisson's ratio for increasing load, and the function of spiral reinforcement. The theory is based upon comparisons between actual test results and predictions based upon computations for schematic concrete structures consisting of equal spheres connected by needle-shaped crystals.

Further notes on the accelerated test for concrete

J. W. H. KING, *Chartered Civil Engineer* (London), May 1957, pp. 15-19

Presents further data (see also November 1955 *Chartered Civil Engineer*) correlating results of compressive strength tests on concrete under accelerated curing within 24 hr with normal 7- and 28-day strengths. (The accelerated curing consists of suggested standard apparatus and procedure for high temperature curing.)

Effect of temperature on the compressive strength of concrete

H. L. MALHOTRA, *Magazine of Concrete Research* (London), V. 8, No. 23, Aug. 1956, pp. 85-94

The crushing strength of concrete at temperatures up to 600 C (approximately 50 percent normal) was independent of the water-cement ratio used but was influenced by the aggregate-cement ratio. Concrete specimens loaded to produce normal design compressive stress during the period of heating showed less reduction in strength than specimens without imposed load. There was a further loss in the compressive strength of the specimens when cooled after heating to a given temperature (approximately 20 percent additional).

Inhibiting alkali reactivity of portland cement

WALTER J. MCCOY (Lehigh Portland Cement Co.), U. S. Patent 2,744,831, May 8, 1956

CERAMIC ABSTRACTS Sept. 1956 (Barbour)

In the art of making concrete structures wherein portland cement containing alkali and a reactive siliceous aggregate are intermixed with water to form a concrete mix and the mix is permitted to set, the method of inhibiting unwanted expansion of the concrete after it has set comprises incorporating in the concrete mix prior to setting a small quantity of a salt selected from the group consisting of chloride, carbonate, fluoride, silicate, nitrate, and sulfate of lithium and sulfate and oleate of copper in an amount not exceeding about 1 percent by weight of the cement but sufficient to inhibit unwanted chemical reaction of the alkali in the cement with the siliceous aggregate after the concrete has set.

Influence of size of concrete test cubes on mean strength and standard deviation

A. M. NEVILLE, *Magazine of Concrete Research* (London), V. 8, No. 23, Aug. 1956, pp. 101-110

AUTHOR'S SUMMARY

It is suggested that the mean strength and the standard deviation of a sample of concrete cubes are functions of the cube size. The results of tests on over 300 cubes of three sizes are given, and from the statistical analysis of these results it is concluded that the smaller 2.78-in. cubes have a significantly higher mean strength and higher standard deviation than the larger 5- and 6-in. cubes.

From these results, it is suggested that the standard deviation obtained from standard 2.78-in. mortar cubes gives an excessive estimate of the effect of the variation in cement quality as compared with the results obtained from 6- or 4-in. concrete cubes.

Practical determination of the quantity of combined water in concrete (in German)

S. BLEIER, *Acta Technica Academiae Scientiarum Hungaricae* (Budapest), V. 13, No. 1-2, 1955, pp. 69-91

HUNGARIAN TECHNICAL ABSTRACTS V. 8, No. 3, 1956

The usual methods for determining the quantity of combined water are cumbersome and the results are unsatisfactory since they also include or may include free, noncombined water. In contrast to this, the method elaborated by the author, based on the specific gravity of concrete, is said to be easily applied in practice and almost free of errors.

Properties of concrete at high rates of loading

D. WATSTEIN, "Symposium on Impact Testing," *Special Technical Publication* No. 176, ASTM, Sept. 1956, pp. 156-170

AUTHOR'S SUMMARY

Effect of the rate of application of load was investigated in compression tests of a concrete having a nominal static strength of 2500 psi. Test specimens were 3 x 6-in. cylinders, and the loads were applied at stress rates ranging from about 10 to 2×10^7 psi per sec. The lower rates of loading were obtained using a conventional hydraulic testing machine, and the higher rates were achieved by loading the specimens in a drop-hammer machine. The duration of impact in the drop-hammer machine was controlled by placing rubber buffers of appropriate thickness and hardness on top of the concrete specimens.

The compressive strength of the concrete increased with the rate of loading. The maximum ratio of dynamic to static compressive strength was about 1.8 for the maximum rate of loading obtained with the drop-hammer machine. There was a significant increase in the secant modulus of elasticity as the rate of loading increased. The resistance of concrete to impact, as measured by its ability to absorb strain energy, also increased with the rate of application of load.

Effect of length to diameter ratio of specimen on the apparent compressive strength of concrete

J. W. MURDOCK and C. E. KESLER, *ASTM Bulletin*, No. 221, Apr. 1957, pp. 68-73

Summarizes previous investigations which indicate that the value of the correction factor also depends on influences other than the l/d ratio. Some nine previous investigations are reviewed and compared with the conclusions that there is little change in the apparent concrete strength when the l/d ratio is between 1.5 and 2.5 and that the correction factor needed must vary according to the strength of the concrete.

It was also noted that the result of tests for which l/d ratio was less than 1 are erratic, indicating that perhaps specimens with these l/d ratios should not be tested. The accepted correction factors favor weaker concretes and penalize high-strength concrete.

Use of 4-in. concrete compression test cubes

A. M. NEVILLE, *Civil Engineering and Public Works Review* (London), V. 51, No. 605, Nov. 1956, pp. 1251-1252

British standard prescribes standard size for compression test cubes as 6 in., but permit the use of 4-in. cubes provided maximum size aggregate does not exceed ¾ in. The paper reports on tests of 4-in. concrete compression test cubes, and recommends their use as standard test cubes whenever the maximum aggregate size is not greater than the ¾ in. allowed, contrary to usual British custom which is the use of 6-in. cubes.

Curing requirements for scale resistance of concrete

P. Klieger, *Bulletin* No. 150, Highway Research Board, Apr. 1957, pp. 18-31

Reports results of a study undertaken to provide additional information to aid in the establishment of minimum ages at which de-icers might safely be used on air-entrained and non-air-entrained pavement. Concludes that no amount of curing will produce a satisfactory resistance to scaling of non-air-entrained concrete. That curing sufficient to develop a necessary strength sufficient to carry traffic loads will be sufficient to develop scale resistance in air-entrained concrete paving.

Studies of "salt" scaling of concrete

G. J. Verbeck and P. Klieger, *Bulletin* No. 150, Highway Research Board, Apr. 1957, pp. 1-13

Presents research data showing amounts of surface scaling related to concentration of salt or other de-icer, curing condition of concrete, air entrainment, and other variables. Principal conclusions are that chemically dissimilar material such as inorganic salts and organic compounds both cause similar "salt" scaling and that the mechanism of scaling is primarily physical rather than chemical. The practical recommendations for scale resistant concrete include air entrainment and a period of air drying before exposure to freezing and thawing.

Testing concrete by measuring speed of a shock wave through it (in French)

Digest No. 809, *Technical Digests*, European Productivity Agency, Paris, No. 8, Aug. 1956, pp. 97-98. (Digested from *Note Technique* No. 9, Institut Technique du Batiment et des Travaux Publics, Paris, 1956.)

Highway Research Abstracts Feb. 1957

By this method, nondestructive tests can be made for hardening, strength, homogeneity, flaws, and effect of frost. The method consists of measuring the velocity of propagation of a shock through the concrete or on its surface by sensitive recording instruments.

Some applications include the following: (1) following the hardening of the concrete at all points of a structure, and the evolution of mechanical strength as a function of time; (2) ascertaining the homogeneity or weak points of a concrete structure and of its strength where it is subjected to the greatest strains; (3) locating junctions which, when badly executed, constitute areas of least resistance; (4) Precise detection of fissures, voids and nests of pebbles within the concrete.

In case of the concrete being attacked by frost, this method permits ascertaining to what depth the frost has penetrated, and at what level the material has remained normal. If the frost has merely retarded setting and consequently hardening, the method will indicate resumption of hardening when the temperature has risen.

Corrosion of chemical origin can be checked exactly, as well as the efficiency of the means used to remedy it.

In the case of reinforced concrete, the method enables fissures and flaws to be detected and proper adhesion of the bars to be checked; combined with the use of a pachymeter, it enables the plan of the reinforcement network to be accurately reconstituted.

Air entrainment in concrete (Luftporenbildende Betonzusatzmittel)

K. Katz, *Bulletin* No. 123, Deutscher Ausschuss für Stahlbeton (Berlin), 1956, 43 pp., 9 DM

Reviewed by H. H. Werner

This is a résumé of the subject of air entrainment in concrete and a comprehensive glossary of international references on the subject.

Investigation of shrinkage and swelling tests for gaseous and foam concrete products (Versuche zur Prüfung der Grösse des Schwindens und des Quellens von Gas- und Schaumbeton)

O. Graf and H. Schaffler, *Bulletin* No. 121, Deutscher Ausschuss für Stahlbeton (Berlin), 1956, pp. 31-39

Reviewed by H. H. Werner

This investigation concerns the standard tests for shrinkage of porous concrete products according to the German Industry Standards (DIN). The standards were silent about the combined variation due to wetting, drying, and shrinkage for such building products. Tests and their results are given, indicating an important factor of swelling due to water absorption while exposed to weather.

Durability of concrete: Physical aspects

Bibliography No. 20, Highway Research Board, 1957, 46 pp., $1

A report by the Highway Research Board committee on durability of concrete — physical aspects—with particular attention to those elements of interest to highway engineers. Durability of concrete exposed to fire, jet blast, and atomic radiation is not included. The first bibliography on durability, *Bibliography* No. 8, was published in 1951. The present publication includes the content of *Bibliography* No. 8 together with supplementary material covering the field through 1956. References are annotated and arranged chronologically.

How can fresh concrete be checked on the building site? (in French)

Digest No. 866, *Technical Digests*, European Productivity Agency, Paris, No. 9, Aug. 1956, pp. 110-111. (Digested from *Note Technique* No. 12, Institut Technique du Batiment et des Travaux Publics, Paris, 1956.)

Highway Research Abstracts
Feb. 1957

Two simple methods of ascertaining on the site the amount of water included in fresh concrete, as well as the proportions of its ingredients, are described.

The quantity of water, usually excessive, is checked by burning.

The relative proportions of the ingredients of fresh concrete are ascertained by weighing in air and water, wet screening to separate fine and coarse aggregates, weighing, and a brief calculation giving four figures corresponding to the four ingredients (water, cement, sand, gravel).

Frost-resistance tests on concretes made with different types of cement

P. D'Elia, *La Ricerca Scientifica* (Rome), V. 25, 1955, pp. 864-882. *Journal of Applied Chemistry* (London), V. 5, No. 9, 1955, p. ii-279

Ceramic Abstracts
Sept. 1956

Concretes made with pure and reduced portland cements, pozzolan, and blast-furnace cements with and without an aerating agent were examined for mechanical properties, e.g., breaking strength and elasticity, and for porosity and permeability. The concrete prepared with pure portland cement had initially the best mechanical properties, but after subjection to repeated freezing and thawing it showed the greatest deterioration.

On the corrosion of concrete (in Finnish)

Tenho Sneck, *Rakennusinsinööri* (Helsinki), V. 12, No. 12, 1956, pp. 143-147

Author's Summary

The influences of carbon dioxide and sulfur dioxide of the atmosphere are briefly discussed. The properties of very pure, soft and hard waters, waters with a low pH value, and the action of waters containing aggressive carbon dioxide are reviewed. The effects of humic compounds, magnesium salts, and sulfates or sulfides are also mentioned. The factors governing the underground corrosion of concrete are discussed with particular attention to the acidity of the soil, the content of exchangeable hydrogen, humic compounds and salts containing sulfur or magnesium. Some dangerous chemicals and especially corrosive environments are reviewed. Some factors influencing the corrosivity of waters and soils are given in the form of a table.

Structural Research

Diagonal shear reinforcing for beams of gaseous and foam concrete (Gas- und Schaumbeton: Schubsicherung von bewehrten Balken)

H. Rusch, *Bulletin* No. 121, Deutscher Ausschuss für Stahlbeton (Berlin), 1956, pp. 1-21

Reviewed by H. H. Werner

A series of tests on diagonal shear reinforcing for porous concrete is described and analyzed. The program was interrupted when other tests showed that the corrosion protection of the reinforcing was insufficient, yet several interesting features were disclosed and are given in this bulletin.

Research on hooped concrete columns (Studium nad slupami uzwojonymi)

A. Kubyllo, *Bulletin* No. 210, Prace Inst. Techn. Budowl., Warsaw, 1954, 18 pp.

Polish Technical Abstracts
No. 1 (21), 1956

The research covered octagonal columns with circular cores, and columns with square sections and square cores. The aim was to demonstrate how changes in sections and in the pitch of the turn influence capacity of hooped concrete columns. When capacity is increased, the greatest part is played by the hooped reinforcement. Concrete resistance may be as low as $R_{28} = 110$ kg per sq cm without affecting column capacity.

Elastic buckling of brittle columns

JOHN C. CHAPMAN and JEAN SLATFORD, *Proceedings*, Institution of Civil Engineers (London), V. 6 (session 1956-57), Jan. 1957, pp. 107-125
Reviewed by ARON L. MIRSKY

Behavior of slender columns or walls which are elastic in compression but brittle in tension was studied theoretically and experimentally. Theoretical relation between failure load and eccentricity was developed, and checked against experimental aluminum alloy column 1.5 x 1 x 66 in., composed of thirty-three 2-in. lengths, stacked but not connected. A comparison with tests on brick walls at the Building Research Station is also given; agreement is fair.

Tests relating to the strength of the moment compression zone (Versuche zur Festigkeit der Biegedruckzone)

H. RUSCH, *Bulletin* No. 120, Deutscher Ausschuss für Stahlbeton (Berlin), 1955, 94 pp., 14 DM
Reviewed by H. H. WERNER

This interesting test program and analysis describes a new approach to the study of the strength of concrete in the compression zone due to bending. The author bases his approach on the similarity between the compression acting in the compression zone of a member subjected to bending and the eccentric compression of a prismatic body. This analogy permits the investigation of elastic behavior in the moment compression zone independent of amount and type of reinforcing and a number of other disturbance factors.

A great number of prismatic, plain concrete specimens were tested and analyzed. Only short-time loading was investigated. Results are illustrated in a great number of graphs. A long-time loading program is planned.

Stress distribution in the zones of the steel load transmission in prestressed pre-tensioned concrete beams of different eccentricities (in German)

H. H. RACKE, *Schweizer Archiv für Angewandte Wissenschaft und Technik* (Solothurn, Switzerland), V. 22, No. 6, June 1956, pp. 169-177
APPLIED MECHANICS REVIEWS Apr. 1957 (Durelli)

Two papers in the field of photoelastic measurements. These papers study in detail the distribution of bond stresses in prestressed, pre-tensioned beams with different eccentricities of reinforcement. The problem is studied assuming a two-dimensional stress distribution. The results obtained are important in the field of reinforced concrete technology but it seems to the reviewer that the most important phase of the papers is the development of new stress analysis materials and techniques.

Tests on compound action between precast reinforced concrete beams and cast-in-place concrete (Versuche ueber den Verbund zwischen Stahlbeton-Fertigbalken und Ortbeton)

O. GRAF and G. WEIL, *Bulletin* No. 119, Deutscher Ausschuss für Stahlbeton (Berlin), 1955, pp. 1-14
Reviewed by H. H. WERNER

Investigated is a series of beams with plain, notched, and indented tops to which a T-flange has been cast later on. An interesting test program of various arrangements is described and analyzed. The results are not fully conclusive nor in accordance with other similar tests but show new features which will be explored in further tests by the same team at the Technische Hochschule Stuttgart.

General

Principle of "convergence-divergence" as a basis for quantitative and qualitative mixing

M. S. FRENKEL, *The Engineers Digest* (London), V. 17: No. 11, 12; 1956; Nov., pp. 475-480; Dec., pp. 509-510
Reviewed by ARON L. MIRSKY

Presents theory underlying new design of continuous crusher and mixer (see also author's article in *Engineering*, July 13, 1956, pp. 41-42; "Current Reviews," ACI JOURNAL, Feb. 1957, *Proc.* V. 53, p. 618).

Concerning specifications for air raid shelters (Betrachtungen zu den Richtlinien fuer Schutzbunker)

THEODOR KRISTEN, *Beton- und Stahlbetonbau* (Berlin), V. 51, No. 11, Nov. 1956, pp. 251-255
Reviewed by VALDIS LAPSINS

Discussion and sampling of engineering aspects of existing regulations for air raid shelters as published by the German State Department for Housing.

New conceptions of strength applied to reinforced and prestressed concrete (Nouvelle conception de la résistance des matériaux)

A. Couard, Génie Civil, Paris, 2nd Edition, 1957, 114 pp., 1250 Fr.

Author's Summary

In this second edition, the author has completed, and modified to a great extent, sections dealing with solid's double functional anisotropy, concrete under torsion-compression, concrete beams under shear, arches loaded at the crown, clay under torsion-compression, due to the conformation, since publication of the first edition, of a second functional anisotropy, resulting from the variable granular interlocking effect. New confirmations deduced from University of Illinois experiments have been added too.

Apart from those important complements to the strength of materials theory, new chapters have been added to evaluate the real security coefficient of some requirements of the French code of practice for reinforced concrete, similar in many ways with those in force in other countries; to expose a new and simplified method of designing thin shells, with simple or double curvature; and a chapter on foundation design and soil investigation.

Cork insulation in forging hammer foundations

The British Constructional Engineer (London), V. 7, No. 4, Mar. 1957, p. 45

Discusses the design of a two-mass foundation designed to provide shock and vibration isolation both for vertical and lateral displacements.

Water towers

J. O. C. Ritchie, *The Structural Engineer* (London), V. 35, No. 1, Jan. 1957, pp. 14-35

Author's Summary

Brief historical notes lead up to 19th century examples followed by the developments to present date, characteristics of design, and estimated costs of reinforced concrete towers with numerous examples designed to meet widely varying conditions and some rarer problems relating to existing towers. Reference is also made to alternative materials of construction.

Concrete for farm construction (in Dutch)

H. de Boer, *Cement* (Amsterdam), V. 8, No. 15-16, Apr. 1956, pp. 388-390

Reviewed by John W. T. Van Erp

Application of concrete for farm construction, cast-in-place as well as prefabricated units, is shown. Advantages are enumerated, especially as far as economy goes in countries where lumber is scarce and its prices are high. Sources of the latest information on concrete farm construction are given.

General aspects of cement grouting of rock

V. L. Mineab, *Proceedings*, ASCE, V. 83, SM 1, Jan. 1957, pp. 1145-1 to 1145-11

Author's Summary

Past experience in the United States of cement grouting of rock foundations is reviewed. There is considerable controversy as to what constitutes best practice in cement grouting; pertinent items are discussed and their pros and cons are presented. Several examples are given of experience on construction projects and of current practices and techniques used in pressure grouting.

Warsaw recording studio

Prefabrication (London), V. 4, No. 40, Feb. 1957 pp. 165-167

The architectural and structural solutions developed in planning a modern sound recording studio, as well as acoustic planning, are discussed. The structural scheme features a space frame composed of two reinforced concrete arches forming a long oval shaped building with a suspended roof structure supported on cables between them. The roof itself is shotcrete.

Winter concreting (in Dutch)

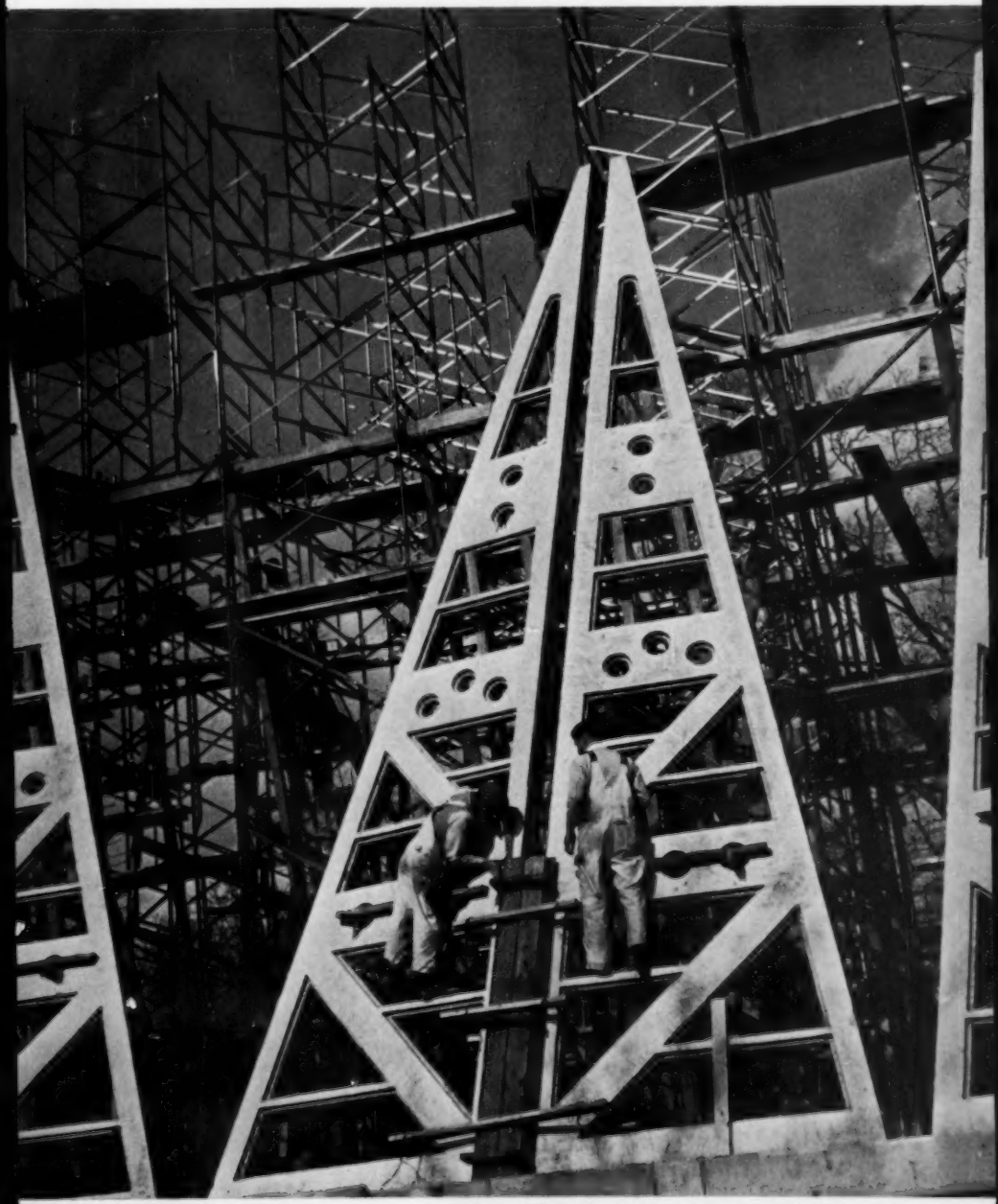
A. A. van der Vlist, *Cement* (Amsterdam), V. 8, No. 21-22, Oct. 1956, pp. 531-537

Reviewed by John W. T. Van Erp

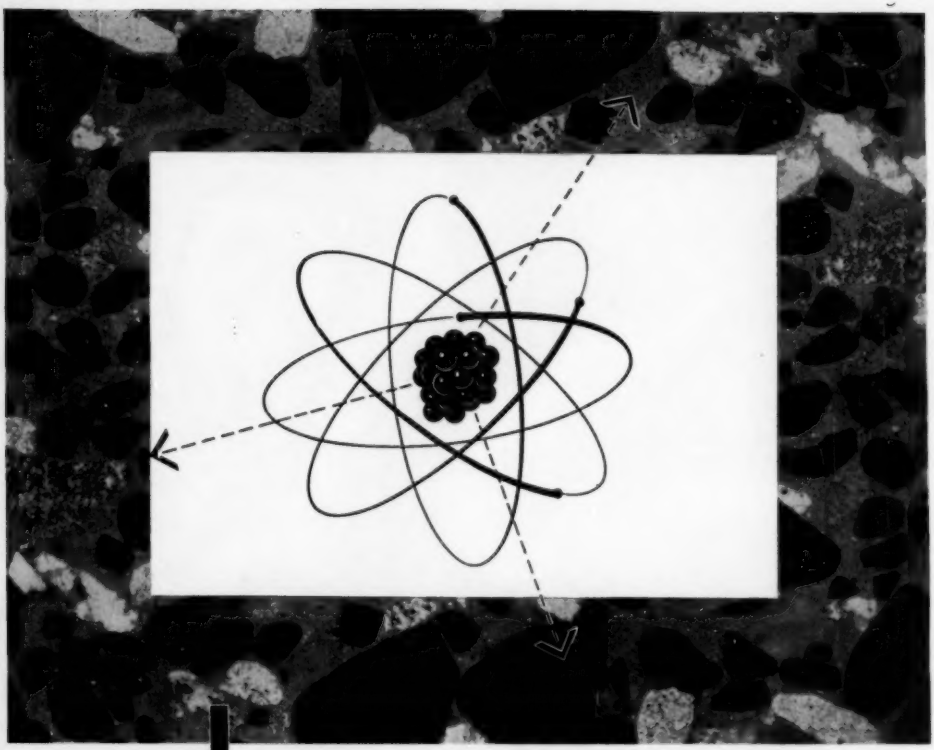
The various measures to be taken are divided into organizational and technical, like the handling of reinforcing steel, cement, aggregates, water, proportioning, mixing, and placement methods. Various types of equipment are described, e.g., heaters, steam generators, and methods of protecting and insulating fresh concrete. Examples of prefabricated elements in winter work are shown.

ACI News Letter

Vol. 29, No. 1 JOURNAL of the AMERICAN CONCRETE INSTITUTE July 1957



Folded plate wall holds stained glass . . . p. 7



"heavy" concrete for reactor shields

High-density concrete, an efficient and practical radiation shielding material, is destined for wide use in nuclear reactors.

The high density results from the use of heavyweight aggregates and affords 1½ to 2 times more radiation protection than normal weight concrete of equal thickness. Its higher unit cost is offset by overall cost savings, which on one project amounted to $7,000,000!

INTRUSION-PREPAKT materials and methods are a proved solution for placing high-density concrete in massive, complex shields having a maze of obstructions. High-density aggregates are carefully preplaced to assure uniform density without segregation. Consolidation with INTRUSION grout produces a PREPAKT concrete structure free of radiation leaks. Densities to 300 lbs./cu. ft. and over can be achieved, usually at less cost than by conventional methods.

INTRUSION-PREPAKT experience in placing high-density concrete reactor shields qualifies it as the top organization to work with on such projects. You will deal with a firm that *specializes* in concrete construction and maintenance, *has* a complete field construction group and *offers* an engineering design service. For information, write: INTRUSION-PREPAKT, Inc., Room 568-G, Union Commerce Bldg., Cleveland 14, Ohio. In Canada: INTRUSION-PREPAKT, Ltd., 159 Bay St., Toronto, Ontario.

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ACI

News Letter Contents

July 1957

On the Cover—Workmen join the precast concrete panels which will serve as support for inch-thick translucent stained glass in the sloping window-walls of the First Presbyterian Church in Stamford, Conn. Story appears on p. 7.

—*Photo courtesy of Harrison and Abramovitz*

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Plans announced for

Seattle Regional Meeting

November 5-6

Plans for the ACI regional meeting to be held in Seattle this November continue to develop as R. H. Lochow, Portland Cement Association, Seattle, chairman of the local committee, announces appointment of committee aides and tentative program plans.

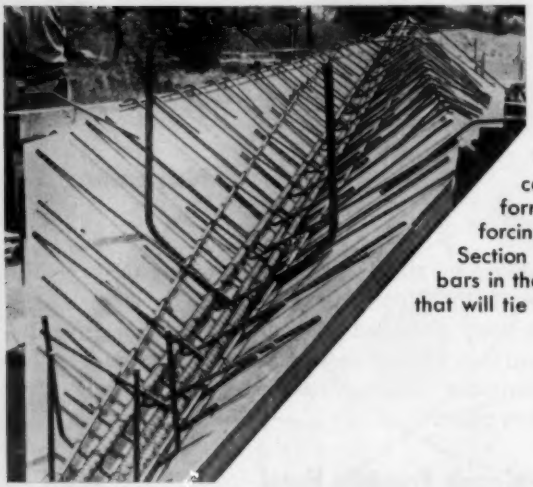
Benjamin Franklin Hotel

The Benjamin Franklin Hotel will serve as headquarters for the meeting. Tuesday, November 5, both morning and afternoon, and Wednesday morning will be devoted to technical sessions. Wednesday afternoon will be reserved for field trips to projects of interest in the Seattle area. Tuesday evening a social hour and banquet will be held at Washington Hotel. Plans for a ladies program will be announced later.

Committee members appointed

R. C. Hildebrandt, Portland Cement Association, Seattle, will be in charge of publicity for the November event, and registration will be handled by Fred H. Rhodes, Jr., University of Washington. A. L. Miller, also of the University of Washington, will supervise educational contact work for the regional meeting.

The finance committee will be headed by Ray A. Young, Charles R. Watts Co., and the technical program will be planned under the leadership of William P. Jackson, Glacier Sand and Gravel Co. Leland Spahr, Concrete Products Association of Washington, is in charge of inspection trips. Donald G. Metcalf, Northwestern Portland Cement Co., will supervise hotel reservation arrangements.



FOLDED PLATE roof is being cast on the site. Casting platform supports two forms. Reinforcing steel partially set in this view. Section is cast upside down. Large bars in the foreground are tie-down bars that will tie into the first-floor corridor walls

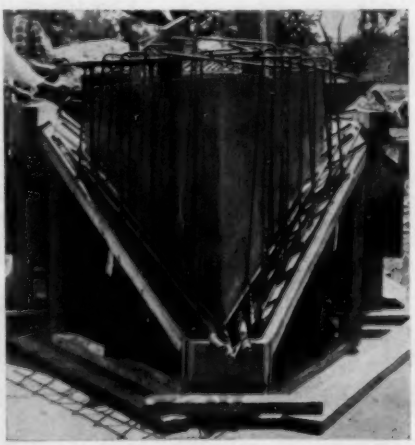
Folded plate roof accents

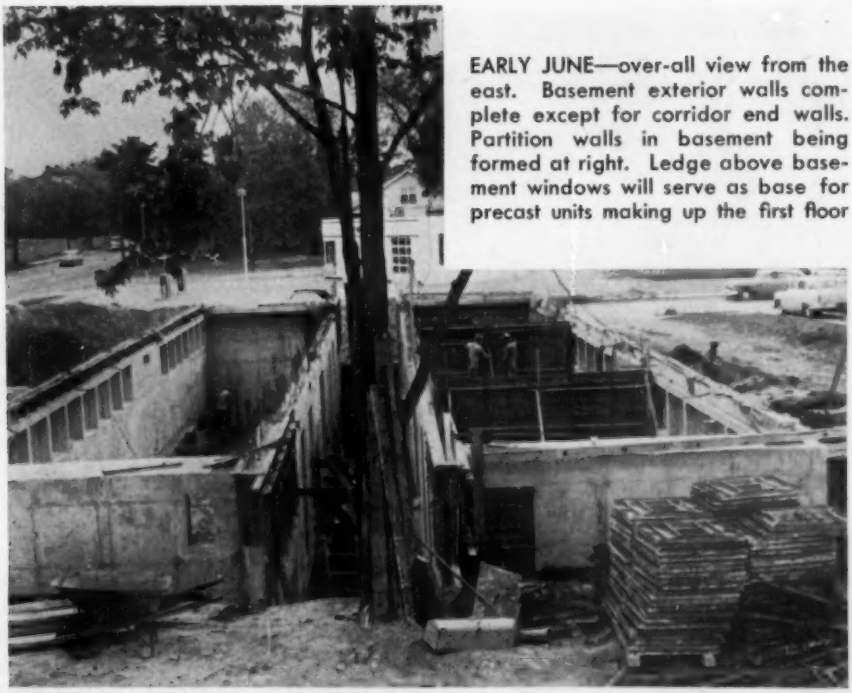
ACI Headquarters

construction scene

PRECASTING AND ERECTION of 46 sections of the unique folded plate roof for ACI's new headquarters neared completion in mid-July, with the contractor following closely his established production schedule. Basement floor slab and first floor of hollow precast units were also completed in July. Principal structural members have all been cast, but miscellaneous concrete work will continue well into August.

Preparations for forming and casting the 4 in. thick "crimped" concrete roof have commanded attention from engineer, architect, and builder alike. Pictured at the top of the page is one stage of construction with reinforcing partly in place; the photograph on the right shows a later stage. Here is a close-up of the portion which will go over the corridor; in addition to part of the "fold" it will include an X-shaped girder system and skylights. The box just behind the vertical steel forms half of the X-shaped beam system.

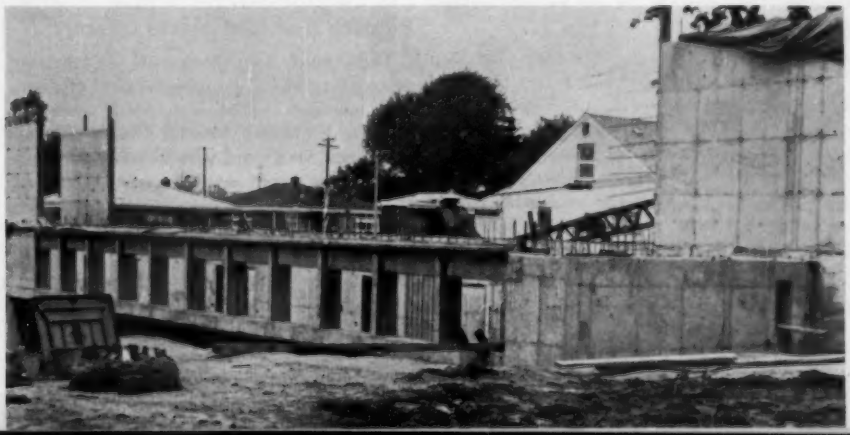




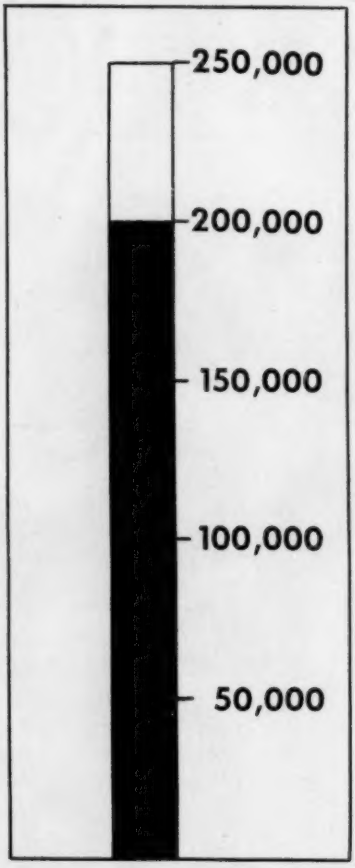
EARLY JUNE—over-all view from the east. Basement exterior walls complete except for corridor end walls. Partition walls in basement being formed at right. Ledge above basement windows will serve as base for precast units making up the first floor

The new Institute headquarters was featured in the May *Architectural Record* in a story on the recent work of Minoru Yamasaki, of Yamasaki, Leinweber and Associates, Royal Oak, Mich. Architect Yamasaki likens the ACI structure to a piece of "jewelry which need have no plain surfaces," referring to the varied design patterns formed in the concrete. Precast elements will form a grill covering basement window openings (see picture below), and another design will appear in the precast grill for the building's

CORRIDOR WALLS above the stairwells at building ends were the first to rise to full building height. Inset portions around basement windows will receive precast grillwork designed to meet esthetic requirements established by the architect



building barometer



end walls. Still a third design in precast concrete will appear in the 6-ft wall encircling the building.

The Detroit office of Pittsburgh Testing Laboratory is selecting concrete mix proportions, and handling testing and inspection of concrete for ACI. Pulte-Strang Inc., Ferndale, Mich., is general contractor; electrical contractor is Southeastern Electrical Co., Inc., Detroit. Kropf Plumbing and Heating and United Heat Engineering Co., both of Detroit, are the mechanical contractors.

Continuing progress in financing headquarters construction has been reported by Henry L. Kennedy, Dewey and Almy Chemical Co., Cambridge, Mass., and members of the Building Committee, which he heads. Contributions by category to the building fund, as of June 15, 1957, are:

Cement producers	$ 66,612.00
Reinforcing steel industry	40,000.00
Engineers and architects in private practice	34,151.22
Contractors	20,445.00
Ready-mixed concrete and aggregate industry	10,833.87
Admixtures manufacturers	7,258.93
Membership at large	4,739.37
Concrete products manufacturers	2,520.00
Concrete machinery and specialty products	2,285.00
Eastern Canada	1,497.40
Special Contributions	1,209.08
Total	191,551.87
Pledges in hand	8,200.00
	$199,751.87

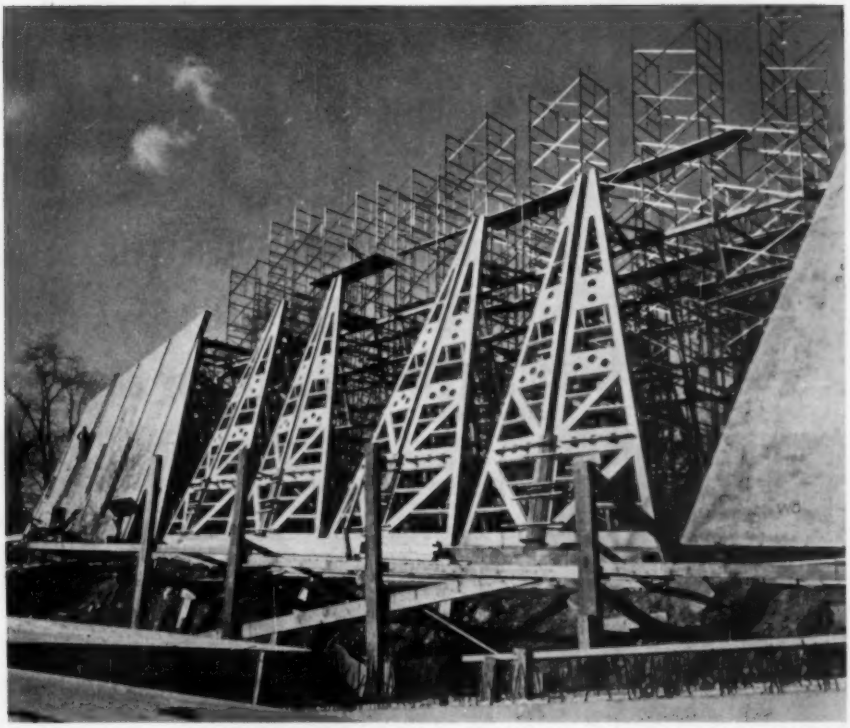
Lount forms new engineering firm

A. Murray Lount, consulting engineer, has announced the formation of the new engineering firm of A. M. Lount & Associates, with headquarters in Toronto. Mr. Lount was associated with the Ontario Hydro-Electric Power Commission for 7 years until 1953, and until recently was a partner in the firm of Lazarides, Lount & Partners.

The new firm will engage in civil and structural engineering work, particularly in heavy civil work including bridges, buildings, docks, substructures, superstructures, and exploration.

Pumice Institute elects officers

The Pumice Institute elected the following officers for 1957 at its annual board of directors meeting in San Francisco: president, William E. Miller, Central Oregon Pumice Co., Bend, Ore.; and vice-president, Rodney Gordenker, Thompson Pumice Co., Tionesta, Cal. Retiring president, R. W. Alley, Jr., General Pumice Corp., Santa Fe., N. M., was elected treasurer, and Lloyd Williamson of Williamson Pumice Co., Bend, Ore., secretary.



Connecticut church uses

PRECAST FOLDED SLABS

PRECAST PANELS weighing up to 10 tons from the sloping walls of the First Presbyterian Church of Stamford, Conn. Similar triangular panels will be placed vertex down in the open spaces visible above. Construction of the building will be achieved without piers or lintels, essentially a folded slab structure, according to structural engineer Felix J. Samuely of London. Scaffolding supports the panels until an 8-in. band of reinforced concrete can be placed between adjacent sections. News Letter cover shows workmen preparing one of the joints for concreting.

Stained glass fills openings in slabs

Openings in the concrete will be filled with inch-thick ruby, amber, emerald, and sapphire colored glass designed by Gabriel Loire and manufactured in Chartres, France. Architect William Harrison, Harrison and Abramovitz, New York, designed the church sanctuary, and his firm cooperated with Sherwood, Mills, and Smith of Stamford in planning other structures in the church plant. The concrete sections are precast on Long Island by Pre-Cast Building Sections, Inc., trucked to the job site, and then erected by crane.

1949 — Walnut Lane Bridge, Philadelphia

1952 — Sunshine Skyway, Tampa, Florida

1955 — Hampton Roads, Norfolk, Virginia

1956 — Havana Sports Stadium, Havana, Cuba

1956 — Litchfield Hospital, Winsted, Connecticut

1956 — Lake Pontchartrain, New Orleans, Louisiana

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Positions and Projects

ACI committee appointments announced

The following members have been appointed to serve on the technical committees listed below:

Committee 208, Bond Stress

S. J. Chamberlin
Iowa State College
Ames, Iowa

ACI-ASCE Joint Committee 323, Prestressed Reinforced Concrete

Lloyd E. Hill
John A. Roebling's Sons Co.
Trenton, N. J.

ACI-ASCE Joint Committee 333, Design and Construction of Composite Structures

Norman W. Hanson
Portland Cement Association
Chicago, Ill.

Committee 605, Hot Weather Concreting

Clayton L. Davis
Universal Atlas Cement Co.
New York, N. Y.

Committee 711, Precast Floor Systems for Houses

Robert A. Rossi
Libbey-Owens Ford Glass Co.
Toledo, Ohio

Forbrich appointed by Pittsburgh Coke

Louis R. Forbrich has been appointed general manager of the Cement Division of Pittsburgh Coke & Chemical Co., Pittsburgh. He joined the firm in 1946.

He is a member of American Society for Testing Materials, Committee C-I on cement, as well as chairman of the Blended Cement Subcommittee, and a member of the Advisory Committee. He has been a member of ACI since 1937.

Building exhibition in London November 13-27

Over 600 exhibitors, representing more than 50 construction industries, will present advanced methods and techniques, as well as new developments in plant and machinery, at the Building Exhibition to be held in Olympia, London, Nov. 13-27, 1957.

The exhibition, which has been held in London every 2 years since 1895, maintains its reputation as a comprehensive display of developments in the building industry, with visitors coming from all parts of the world. All British government organizations concerned with building take part in the exhibition, and professional and trade associations also are active participants.

Adams changes position

Robert F. Adams, formerly with the U. S. Bureau of Reclamation, Denver, is now senior materials and research engineer for the California State Department of Water Resources, Sacramento.

IMPORTANT

ACI Regional Meeting

SEATTLE

Benjamin Franklin Hotel

scheduled for

November 5-6, 1957

Please note date

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features functional

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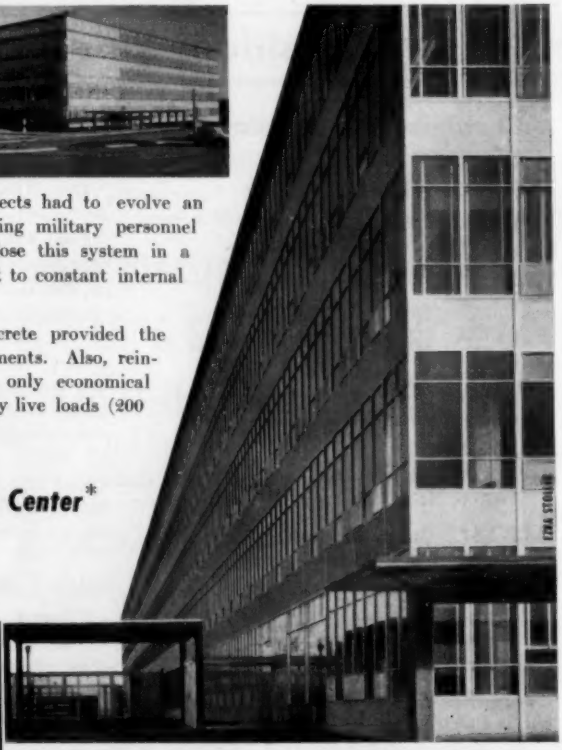
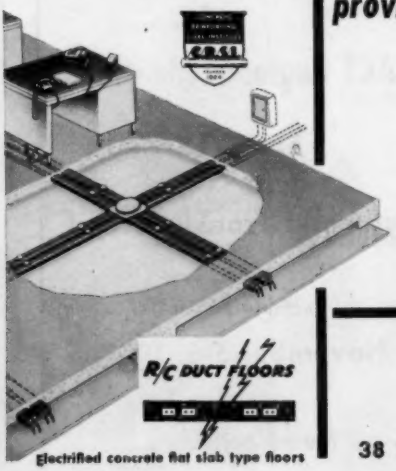
*St. Louis, Missouri





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Eney awarded top faculty honor

Prof. William J. Eney, head of the civil engineering department, received Lehigh University's top faculty award at the tenth annual faculty dinner. Eney was named recipient of the Hillman Award as "the member of the Lehigh faculty who has done the most toward advancing the interests of the university."

Since coming to Lehigh in 1936, Professor Eney has been responsible for initiation of a varied research program including pioneer work in prestressed concrete at Fritz Engineering Laboratory, which he also heads.

Penniman & Browne expand

Penniman & Browne, Inc., has announced the opening of its new engineering and research division in Baltimore. Laboratory and field facilities for physical testing, test boring, and control testing of concrete, soils, and asphalt are provided.

Plummer moves to American Welding Society

Fred L. Plummer has accepted the position of national executive secretary and technical director of the American Welding Society, New York. He formerly was director of engineering of Hammond Iron Works, Warren, Pa

Koontz to head department at Worcester Polytechnic Institute

Prof. Carl H. Koontz has been appointed head of the civil engineering department at Worcester Polytechnic Institute, Worcester, Mass. He joined the faculty as an assistant professor in 1952.

Connolly and Harbison elected; Noble retires from Hercules

Charles H. Noble, Jr. has retired as treasurer of the Hercules Cement Corp., Philadelphia, and Leonard P. Connolly has been elected as treasurer of the company. George M. Harbison has been chosen controller.

Glantz promoted by Penn-Dixie

O. J. Glantz, superintendent of the Penn-Dixie Cement Corp. plant at Petoskey, Mich., has been promoted to the position of director of research with headquarters at Nazareth, Pa. Mr. Glantz has been with Penn-Dixie since 1953.

Egyptian engineer on tour

M. Kamel Zeitoun, director of architecture section, Building Research and Training Center, Giza, Egypt, visited ACI headquarters in June.

Mr. Zeitoun is in the United States on a year's visit to plan future training courses for Egyptian engineers, technologists, and mechanics in construction and building material production.

While in Detroit he observed autoclave curing of lightweight concrete block and central mix and dry batch ready-mixed concrete operations, and visited the new ACI headquarters building site.

Wanted . . .

ACI Proceedings

V. 10 (1914)

V. 11 (1915)

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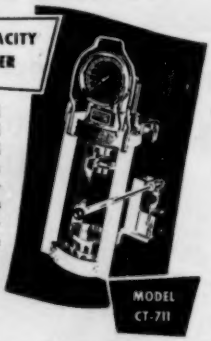

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Granco promotes Tiarks

F. Alan Tiarks has been promoted to general sales manager of the Granco Steel Products Co., St. Louis. He joined Granco in 1950 in the engineering department.

Ideal Cement orders self-unloading bulk cement barges

Ideal Cement Co. of Denver recently contracted with Todd Shipyards Corp., Seattle, for design and construction of two self-unloading bulk cement barges to be delivered in December, 1957. The hulls of the non-self-propelled barges will be of all-welded steel construction with nominal dimensions of 170 x 44 x 12 ft. Rapid unloading of the 7500-bbl hoppers in each of the carriers will be by the F-H Airslide conveyor produced by Fuller Co.

Roberts named Intrusion-Prepakt vice-president

Appointment of Harrison H. Roberts as vice-president of Intrusion-Prepakt, Inc. and the Prepakt Concrete Co., Cleveland, was recently announced. In this position Mr. Roberts will deal with all phases of operations throughout the world. He joined Intrusion-Prepakt in 1956 as manager of foreign operations.

REVIEWERS NEEDED

POLISH—*Archiwum Inzynierii Ladowej*

ENGLISH—*Civil Engineering and Public Works Review*

A volunteer reviewer is needed for each of these monthly magazines to keep members informed of pertinent papers in them through the "Current Review" section of the JOURNAL. Those interested should write the Secretary of the Technical Activities Committee at ACI headquarters.

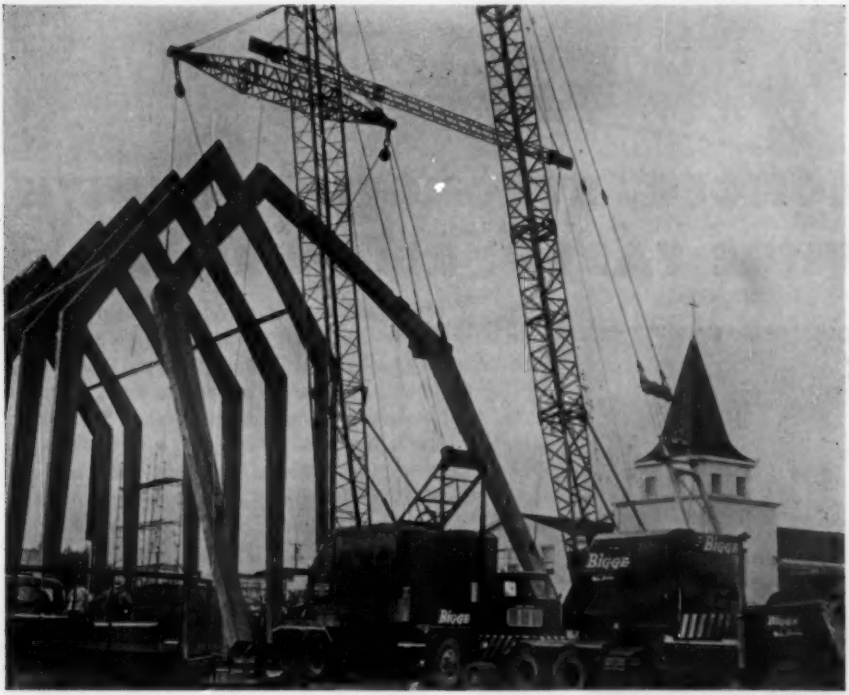

Photo from Engineering News-Record, 12-27-56

58-FT HIGH PRECAST BENTS, perhaps the highest ever precast, were hoisted into place to form the frame for the First Presbyterian Church of San Mateo, Calif. The units weighing up to 40 tons were cast flat and erected by two cranes. Cracking was prevented by turning from horizontal to vertical position with the member off the ground. Architect is Donald Powers Smith and structural engineer is Isadore Thompson, both of San Francisco. General contractor is Trewitt & Shields of Fresno. Erection was by Bigge Crane & Rigging Co., of Oakland.

Narver establishes scholarship for engineers at Stanford

A perpetual endowment scholarship for civil engineering students has been established at Stanford University, Stanford, Calif., by Mr. and Mrs. D. Lee Narver. Mr. Narver is board chairman of Holmes & Narver, Inc., Los Angeles engineers and constructors. The $20,000 scholarship will provide full tuition for one student or partial tuition for two. To be eligible for the scholarship applicants must be male residents of southern California in definite need of aid. The scholarship may be used for fifth and sixth years of study if it has carried the student through his undergraduate work.

Taiwan Cement executives visit ACI headquarters

Two executives of Taiwan Cement Corp., Taipei, Taiwan, Mr. Lim, chairman of the board of directors, and Dr. H'su, director of the organization, visited ACI headquarters in Detroit recently.

They are on tour of the United States to study and observe American practices in cement making and use of cement in concrete products, paving, and housing.

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Who's Who This Month

ACI Committee 214

"Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-57)," p. 1, adopted June 1 as an ACI Standard, is the work of ACI Committee 214. W. A. Cordon, Utah State Agricultural College, Logan, Utah, was chairman of the committee which prepared the standard.

Members of the committee were: Edward A. Abdun-Nur, consulting engineer, Denver; Delmar L. Bloem, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D. C.; Elwood H. Brown, University of California, Berkeley; T. G. Clendenning, Hydro-Electric Power Commission of Ontario, Toronto; Herbert K. Cook, Master Builders Co., Cleveland; Louis A. Dahl, Portland Cement Association, Chicago; R. J. Elfert, U. S. Bureau of Reclamation, Denver; J. A. Kauer, Huron Portland Cement Co., Detroit; Thomas B. Kennedy, Waterways Experiment Station, Jackson, Miss.; J. D. Lindsay, Division of Highways, Bureau of Materials, Springfield, Ill.; Edmund A. Pratt, consulting engineer, Berkeley Heights, N. J.; Horace A. Pratt, University of Maine, Orono; and Charles S. Whitney, Ammann & Whitney, Milwaukee and New York.

Herbert A. Sawyer, Jr. and Jack E. Stephens

"Under-Reinforced Concrete Beams under Long-Term Loads," p. 21, is the work of Herbert A. Sawyer, Jr. and Jack E. Stephens.

Herbert A. Sawyer, Jr. is associate professor of civil engineering at the University of Connecticut, Storrs. He graduated in civil engineering from Lafayette College, Easton, Pa., in 1939. Following part-time teaching and graduate work leading to a master's degree at Yale University, he was employed by the Chicago Bridge and Iron Co., as a civil engineer. During most of this period he was engineer in charge of construction of floating dry-docks for the navy, for which he received the Meritorious Civilian Service Award of the Bureau of Yards and Docks and the professional CE degree from Lafayette College. Professor Sawyer returned to teaching in 1946 and has taught structural engineering at Lehigh University, the University of Alabama, and, since 1950, the University of Connecticut.

A member of ACI since 1948, Professor Sawyer is also a member of ASCE, the American Society for Engineering Education, and other professional societies. He is author of papers and discussions including several in the field of ultimate load and limit design.

Jack E. Stephens, assistant professor of civil engineering, University of Connecticut, graduated from the University of Connecticut with a BS in engineering in 1947, and received the MS degree from Purdue in 1955. After teaching at the University of Connecticut for 1½ years, he spent 2 years as a junior highway engineer with the Connecticut Highway Department. Assignments during this period included inspection of the concrete work on the Commander Hull Bridge and associated structures in Derby, Conn.

Mr. Stephens assumed his current teaching and research duties in 1950. He is a member of ASCE and Connecticut Society of Civil Engineers, among other professional affiliations.

John F. Brotchie

ACI member John F. Brotchie has prepared "General Method for Analysis of Flat Slabs and Plates" appearing on p. 31 of this month's JOURNAL.

Mr. Brotchie graduated in civil engineering at Melbourne University, Australia, in 1950. Since then he has served with the Commonwealth Department of Works, Head Office, Melbourne. He spent his first year with the department on the design of roads and airfields, then turned his attention to the design of structures in steel, reinforced concrete, and prestressed concrete. He is now

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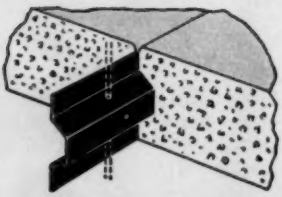
TONGUE AND GROOVE JOINT



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senior structural engineer in charge of the design of major structural projects.

Since graduation, Mr. Brotchie has pursued advanced studies in soil mechanics, plastic design, prestressed concrete, and engineering mathematics. The theory described in his current paper has also been extended to general elastic analysis and to plastic analysis.

Warrington G. Mitchell

Warrington G. Mitchell, concrete technician, Mobile District, Corps of Engineers, U. S. Army, is author of "Significant Effects of a Waterproof Coating on Concrete Durability" appearing on p. 51.

After attending Rutgers University, Mr. Mitchell was engaged primarily in field survey and layout on construction from 1928 to 1935. He then joined the North Jersey Quarry Co. as an analyst working on control testing of aggregates and proportioning for ready-mixed concrete plants.

In 1941 he joined the Corps of Engineers as chief concrete inspector on construction of Stewart Airport and West Point Military Academy. Later he transferred within the Corps of Engineers to the Central Concrete Laboratory, North Atlantic Division, and served as assistant to the chief of the Concrete Mixture Design Section. In 1945 he became chief of this section continuing in this position until 1949. At this time he transferred to the South Pacific Division as concrete engineer on the construction of the Pine Flat Dam on the Kings River in California.

In 1952 Mr. Mitchell resigned from the Corps of Engineers to take a position with the Indian government as concrete engineer on Bhakra Dam, Punjab State, India. Joining the consulting engineering firm of Tippetts-Abbett-McCarthy-Stratton in 1953, he served as concrete engineer on the construction of the Seyhan Dam, on the Seyhan River near Adana, Turkey; and on the Kemer Dam in western Turkey. In 1956 he returned to the United States and rejoined the Corps of Engineers in his present capacity.

He is a member of ACI and the Society of American Military Engineers.

G. A. Mansfield, C. A. Sirrine and Benjamin Wilk

A trio of authors, G. A. Mansfield, C. A. Sirrine and Benjamin Wilk, presents "Control Joints Regulate Effect of Volume Change in Concrete Masonry" which appears on p. 59.

ACI member G. A. Mansfield is general manager of Wm. Moors Concrete Products, Inc., Fraser, Mich. He received his degree in civil engineering from the University of Detroit in 1930. Following graduation he was with the research and testing division of the Michigan State Highway Department. In 1942 he joined Huron Portland Cement Co. and was technical service director and division sales manager, when he left that organization in 1948 to accept his present position with a manufacturer of lightweight concrete masonry units and precast floor and roof members.

C. A. Sirrine, executive secretary, Concrete Products Association of Michigan, Detroit, received his BS in architecture from the University of Michigan in 1926. A member of the Michigan Society of Architects and the American Institute of Architects, Mr. Sirrine was formerly a partner in the firm Loree & Sirrine, Ann Arbor, Mich. He assumed his present duties in 1950. An ACI member, he serves on Committee 331, Structures of Concrete Masonry Units, and is a member of the Concrete Improvement Board of Detroit.

Benjamin Wilk, vice-president and general manager of Standard Building Products Co., Detroit, received his CE degree from the University of Minnesota in 1914. Upon graduation he became associated with the city engineering department of Virginia, Minn., following which he spent a year with the Interstate Commerce Commission in valuation work. In 1916 he joined Universal Portland Cement Co., Chicago, as an engineer in the technical service bureau, where he remained until 1926 when he left Universal to become

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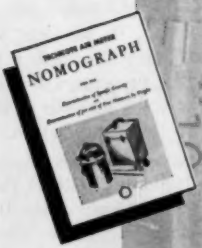

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manager of Standard Building Products Co. He is a past-president of the National Concrete Masonry Association, and now heads the Concrete Products Association of Michigan. Mr. Wilk has taken a keen interest in Institute activities and was a member of its Publications Committee for a number of years. He has contributed several papers for publication in the JOURNAL, and is an active member of ASTM Subcommittee C 15.

Honor Roll

February 1–May 31, 1957

G. B. Southworth is still leading the Honor Roll with 13 credits. Arthur N. L. Chiu is second with 7 and Phil M. Ferguson and Walter H. Price are tied in third place with 5 credits each.

A member may benefit greatly from ACI's reservoir of technical information and the new member YOU propose will certainly bring important information to ACI.

G. B. Southworth	13
Arthur N. L. Chiu	7
Phil M. Ferguson	5
Walter H. Price	5
Luis Perez Cid	4
Jaime de las Casas	4
Louis A. Gottheil	4
Martin J. Gutzwiller	4
Henry L. Kennedy	4
L. M. Legatski	4
James A. McCarthy	4
H. C. Pfannkuche	4
James C. Amrhein	3
Nicandro Barboza	3
C. A. Engman	3
Ambrosio R. Flores	3
J. E. Jellick	3
Franklin B. Johnson	3
I. Martin	3
Martin Schulz	3
Howard Simpson	3
H. T. Williams	3
John T. Young	3
Miles N. Clair	2½
W. H. Armstrong	2
R. F. Blanks	2
James Chinn	2
Dale Cobb	2
W. S. Cottingham	2
Roger D. de Cossio	2
E. I. Fiesenheiser	2
Kenneth H. Gedney	2
N. L. Hinkson	2
M. W. Huggins	2
Leo Liberthson	2
Robert H. Lochov	2
Patrick McNally	2
Frederick T. Mavis	2
Izuo Miyashita	2
George H. Nelson	2
Richard A. Parmelee	2
Abdur Rahman S. Rasul	2
Harry Saxe	2
William C. Schwenger	2
Ellis S. Vieser	2

New Members

The Board of Direction approved 57 Individual applications, 2 Corporations, 15 Juniors, and 9 Students, making a total of 83 new members. Considering losses due to deaths, resignations, and nonpayment of dues, the total membership on June 1, 1957, was 8830.

Individual

ARMSTRONG, ELLIS L., Massena, N. Y. (Asst. Prof. Mgr., St. Lawrence Power Project, Uhl, Hall & Rich)

BEALL, RALPH D., New York, N. Y. (Sr. Struct. Engr., Western Electric Co., Inc.)

BECKHAM, CHARLES W., Ft. Worth, Texas (Cons. Struct. Engr.)

BIGGS, HOWARD M., Washington, D. C. (Vice Pres., R. H. H. Spidel, Inc.)

BROADUS, BILLY JACK, SR., Beaumont, Texas (Cons. Struct. Engr.)

BURKHART, EDSEL J., Bryan, Texas (Sec.-Treas., Prin. Engr., Spencer J. Buchanan & Assocs., Inc., Cons. Engrs.)

CALDWELL, DAVID, Auckland, New Zealand (Asst. Engr., Auckland Harbour Board)

CAMARGO, NELSON DE BARROS, Sao Paulo, S. P., Brazil (Cons. Struct. Engr.)

CHAPMAN, P. G., Dallas, Texas (Gen. Mgr., Preston Co.)

CLARKE, TERENCE K., Cleveland, Ohio (Cons. Struct. Engr.)

DARROW, DONALD C., Odessa, Texas (Sr. Partner, Darrow-Hodgens-Harp, Archs. & Engrs.)

DEARRIETA, JOHN, Canoga Park, Calif. (Design Engr., Atomics International, Div. of North American Aviation, Inc.)

DECORY, LANCE C., Roswell, New Mexico (Field Engr., Portland Cement Association)

ESTRADA-GOMEZ, ENRIQUE, Ciudad Trujillo, Republica Dominicana (Engr., Hwy. Dept., Secretaria de Obras Publicas)

FONG, HEOU-SAN, New York, N. Y. (Bridge Engr., Taiwan Hwy. Bureau, The Preload Co.)

GAFFNEY, WILLIAM P., Teaneck, N. J. (Concrete Engr., Andrews & Clark)

GOLDBECK, ALBERT T., Washington, D. C. (Engrg. Consultant, National Crushed Stone Assoc.)

GWIN, LEWIS L., Altoona, Pa. (Pres., Gwin Engrs., Inc.)

HALL, KENNETH W., Raleigh, N. C. (Mgr., Raleigh office, Froehling & Robertson, Inc.)

HALPERN, IRVING W., Ottawa, Ont., Canada (C. E., Dept. National Defence)

HIROTA, ICHIRO, Tokyo, Japan (Technician, Maeda Kensetsu Kogyo Co.)

HUNTER, CALVIN K., Hatboro, Pa. (Matls. Engr., The Liberty Corp.)

KHAN, AMINULLAH, Lahore, Pakistan (Irrigation Engr., Irrigation Dept., West Pakistan Gov't)

KLINE, EUGENE R., Cincinnati, Ohio (Sales Engr., Marquette Cement Mfg. Co.)

KNEE, THOMAS L., Trenton, N. J. (Struct. Engr. & Arch., State of N. J.)

KNIGHT, CURTIS, Port of Spain, Trinidad, B. W. I. (Design Engr., Gov't of Trinidad, Works & Hydraulics Dept.)

KULKA, FELIX, Los Angeles, Calif. (Assoc. Engr., T. Y. Lin & Assocs.)

LAVELLE, FRANCIS H., Providence, R. I. (Sr. Struct. Engr., Charles A. Maguire & Assocs., Engrs.)

LESLIE, HOWARD A., Homestead, Pa. (Pres., West Penn Testing Laboratories, Inc.)
LONG, GORDON L., Sacramento, Calif. (Head, Hydraulics Engrg. Laboratories Branch, California Dept. of Water Resources)
LOVETT, MARTIN, New York, N. Y. (Struct. Engrg. Consultant)
MARABLE, JAMES R., Savannah, Ga. (Sr. Engr., Union Bag-Camp Paper Corp.)
MARBAN ALVAREZ, JOSE, Manzanillo-Oriente, Cuba (Bldr.)
MARCUS ROSEN, SIMON, Mexico, D. F., Mexico (Concrete Constructions)
MARIN BERMUDEZ, ALBERTO, Caracas, Venezuela
MCMULLEN, O. E., Lansdowne, Pa. (Cons. Engr.)
MOGA, NICOLAS I., West Palm Beach, Fla. (Struct. Engr., Marine Construction & Engrg. Co.)
NOYAK, ROBERT L., Colorado Springs, Colo. (Chf. of Lab., Tech. Asst., Skidmore Owings & Merrill)
OGLE, ARLIE E., New Orleans, La. (Cons. Engr.)
PENNACHETTI, JOHN T., Thorold, Ont., Canada (Pres., Thorold Concrete Block Co., Ltd.)
PERKINS, W. M., Memphis, Tenn. (Engr., Sr. Planner, Memphis & Shelby County Planning Comm.)
RANDALL, FRANK A., JR., Chicago, Ill. (Struct. Engr. PCA)
RENALL, IAN GARTH, Wairarapa, North Island, New Zealand (Renall Bros., Bldg. Contractors)
REYNOLDS, KENNETH B., San Francisco, Calif. (Chf. Engr., Engrg. office of Clyde C. Kennedy)
RIBYAT, SEYMOUR, Minoa, N. Y. (Cons. Engr.)
ROSLING, NIELS E., Charlottenlund, Denmark (Design Engr. & Field Inspector, Rasmussen & Schiotz)
ROWLEY, JOSEPH D., Kingsport, Tenn. (Gen. Mgr., Super. Constr., Bays Mountain Const. Co., subsidiary of Eastman Kodak Co.)
SARDELLA, FRANCESCO, Milano, Italy (TECNIMASTER, Dist. of The Master Builders Co. products of Italy)
SCHLESINGER, LEE, Chicago, Ill. (Chf. Struct. Engr., Western-Knapp Engrg. Co.)
SHINKLE, ROBERT W., Glendora, Calif. (Design Engr., Aerojet General Corp., Azusa)
SHOUKRY, ZAHER, Columbia, Mo. (St. at Univ. of Mo.)
SMITH, CHARLES RICHARD, Cambria, Va. (Struct. Engr., Asst. Prof. of Struct. Design, Virginia Polytechnic Institute)
SOLEM, JAY L., Royal Oak, Mich. (Struct. Engr. Giffels & Vallet, Inc., Detroit)
STEELE, ROBERT, Topeka, Kansas (Struct. Engr. Squad Leader, Burgwin and Martin)
VERVAKE, ALBERT J., JR., Roseville, Mich. (Struct. Engr., Giffels & Vallet, Detroit)
WILLIAMS, ROBERT R., Ndola, N. Rhodesia (Design Engr., Menzies & Durkin)
WRAY, NELSON I., Woodbury, L. I., N. Y. (Consultant, PCA)

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ARAVIND KESHAVLAL, KHETIA, New York, N. Y. (Struct. Engr., Ammann & Whitney)
BUTUN, HILMI R., Detroit, Mich. (Bridge Designer, Harley, Ellington & Day, Inc.)
CASSIDY, A. B. B., Kitimat, B. C., Canada (Jr. Inspector, Coast Testing Labs., Ltd.)
COHEN, MARVIN B., Atlanta, Ga. (Struct. Designer, John J. Harte Co.)
FURST SEGUIN, MIGUEL MARIA, Porlamar, Estado Nueva Esparta, Venezuela (Concrete Design Engr., Construction y Topografia)
GONZALEZ A., CARLOS, Cambridge, Mass. (M. I. T.)
HAMMEN, WALTER B., Baltimore, Md. (Draftsman, Benj. E. Beavin Co.)
HOLTE, ROY, Woodbury, N. J. (Bridge Designer, Brown & Blauvelt)
MONAHAN, THOMAS, Chicago, Ill. (Draftsman, Bethlehem Steel Co.)
NEAIMY, HAZIM AL, Baghdad, Iraq (Engrg. College)
SHORE, MALCOLM ROBERT, Auckland, New Zealand (Engrg. Grad., Auckland Harbour Bd.)
SMITH, JAMES E., Michigan City, Ind. (Design Engr., Boyd E. Phelps, Inc.)
STRAGIER, MARCEL G., Barstow, Calif. (Chf. Engr. & Admin. Asst., Public Works Branch, U. S. M. C. Supply Center)
WONG LOPEZ, FELIX LEON, Anzoategui, Venezuela (Jr. C. E., Mene Grande Oil Co.)

Student

ALCOCK, DENIS H., W. Lafayette, Ind. (Purdue Univ.)
BISHOP, L. JAMES, Walled Lake, Mich. (Univ. of Mich.)
GUPTA, S. SHANKAR, Champaign, Ill. (Univ. of Ill.)
HAUKE, JEROME CHARLES, Milwaukee, Wis. (Univ. of Wis.)
HERMOSILLA, JUAN JOSE, Guatemala City, Gautemala (M. I. T.)
KENDALL, FRED M., Chico, Calif. (Chico State College)
KENNEDY, NORMAN E., Mansfield, Mass. (Jr. C. E., Commonwealth of Mass., Dept. of Public Works)
LEE, ROBERT HAROLD, Homewood, Ill. (Purdue Univ.)
TRAVIESO CARDONA, ANTONIO, Caracas, Venezuela (Universidad Catolica Andres Bello)

LOOKING AHEAD

July 29-August 2, 1957—World Conference on Prestressed Concrete, Fairmont Hotel, San Francisco, Calif.

August 12-24, 1957—International Society of Soil Mechanics and Foundation Engineering, London, England

August 19-30, 1957—(Tentative) Seventh Pan American Highway Congress, Panama City, Panama

September 8-13, 1957—American Chemical Society, National Meeting, New York, N. Y.

September 9-13, 1957—Instrument Society of America, 12th Annual Instrument-Automation Conference and Exhibit, Auditorium, Cleveland, Ohio.

September 23-25, 1957—Standards Engineers Society, Sixth Annual Meeting, Hotel Commodore, New York, N. Y.

October 9-11, 1957—National Slag Association, 40th Annual Meeting, Plaza Hotel, Washington

November 5-6, 1957—ACI Regional Meeting, Benjamin Franklin Hotel, Seattle, Wash.

Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. The grades of membership are described overleaf.

All who have an interest in concrete are eligible for membership.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI JOURNAL provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

Opportunity for service is present in technical committee activity, in contributions, or only comments, to the ACI JOURNAL, or in reviewing technical publications for material of interest to the membership.

ACI's world-wide membership is growing in extent and participation—traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

(cut here)

Board of Direction, American Concrete Institute
P. O. Box 4754, Redford Station
Detroit 19, Michigan

Date_____

Individual Members (U. S. and Possessions, Canada, Mexico, Central America, and West Indies)	$15.00
Individual Members (All other foreign countries)	12.00
Corporation Members	50.00
Contributing Members	100.00
Junior Members—nonvoting (under 28)	7.50
Student Members—nonvoting (under 28)	5.00

(Subject to stipulations of Bylaws—Article I on reverse side. Bylaws on request.)

Of the annual dues, $12.00 is for the JOURNAL of the American Concrete Institute (except that dues for Junior and Student Members apply in full for the JOURNAL).

The undersigned hereby applies for_____**membership**
(Individual, Corporation, Contributing, Junior, Student)

in the American Concrete Institute. Proposed by_____

For Corporation Membership, ACI representative will be_____

_____ _____
(Date of graduation if Student) (Name, if Corporation)

Signature_____

For our records, please complete both sides of application.

EXCERPTS FROM BYLAWS: ARTICLE I—MEMBERS

Section 1. This Institute shall consist of Members, Corporation Members, Contributing Members, Junior Members, Student Members, and Honorary Members interested in furthering the Institute's objects as set forth in its Charter.

Sec. 2. A Member shall be an individual. A Corporation Member shall be a firm, corporation, society, agency of government or other organization. A Corporation Member shall name one individual as its representative who will enjoy all membership rights and privileges.

A Contributing Member shall be an individual, firm, corporation, society, agency of government, or other organization wishing to give larger support to Institute activities through the payment of larger dues. Contributing Members, other than individuals, shall name representatives as do Corporation Members.

A Junior Member shall be a person less than 28 years old.

A Student Member shall be less than 28 years old and a student in residence at a recognized technical or engineering school.

An Honorary Member shall be a person of eminence in the field of the Institute's interest, or one who has performed extraordinarily meritorious service to the Institute.

Sec. 3. All classes of Members except Honorary Members and Student Members, shall be proposed by at least one Member of the Institute and elected by a two-thirds vote of the Board of Direction. An Honorary Member shall be elected by unanimous vote of the Board of Direction. A Student Member shall be proposed by a member of his school's faculty, who need not be a Member of the Institute, and elected by a two-thirds vote of the Board of Direction.

Sec. 4. All Members shall have all rights and privileges of membership as determined by the Board of Direction except that a Junior or Student Member shall not vote nor hold office. The status of a Student Member shall change automatically to that of Junior Member on the first anniversary of his membership succeeding the date on which he ceases to be a student in residence. The status of a Junior Member shall be changed to that of Member on the first anniversary of his membership after he becomes 28 years of age.

Sec. 5. Applications for and resignations from Membership and requests for change of representatives of Corporation or Contributing Memberships shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from Members whose dues are not more than 60 days in arrears, except by special action of the Board of Direction.

(cut here)

Name _____
Please Print

Date of Birth _____
Month Day Year

Title or Position _____

Name of Firm or Organization _____

☐ **Business Address** _____

☐ **Resident Address** _____
(Please check address to which you wish mail and publications sent)

Nature of Firm's Business _____

The ACI Membership Directory will be sent—as available—only on request. Check here if you wish to receive the latest edition. ☐

Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Portable bulk plant

A 36-ft-tall portable bulk plant has been developed which can unload a full railroad hopper car of pulverized material in less than 2 hr. The triple section plant provides "dead storage" for 7500 cu ft of dry granular products and can load it aboard trucks at the rate of about 20 cu ft per min through airslide spout mounted on middle unit's side. Bulk plant unit may be quickly moved from one site to another by fastening detachable wheels to one end and towing it behind truck-tractor.—Delta Tank Manufacturing Co., Inc., Baton Rouge, La.

Forms simplify roof construction

Metal form for use in waffle-type concrete joist construction is quickly blasted free of the hardened concrete with a jet of compressed air, leaving smoothly-patterned ceiling surfaces. With Steeldome forms, one man presses an air gun against a fitting set into the form, while another man stands by to grasp the form as it is blasted free.

Designed especially for exposed ceiling construction the Ceco form is a rigid, deep-drawn, one-piece unit that makes it possible to construct economical wide-open floor areas with no projecting beams. Flanges forming standard-width joist soffiits are an integral part of the domes. When erected the forms are butted at all soffit joints, eliminating the common problems of forms which must be lapped, and creating the smooth surfaces desired for exposed ceilings. The rigid, integral soffit flanges permit skeleton centering in one direction instead of two. If attached or suspended ceilings are required ceiling hangars are installed through holes provided in the metal dome.—Ceco Steel Products Corp., 5601 W. 26th St., Chicago 50, Ill.

MARACONS are water-reducing admixtures for concrete. They permit more complete hydration of cement particles and permit a substantial reduction in the unit water content without loss of plasticity or consistency of the mix. This means —

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1. Attain higher strengths without increasing cement content of a mix.
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The MARACONS also reduce water requirements in concrete mixes containing pozzolanic materials.

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Errata

The following correction should be made in "Mortar- and Concrete-Making Properties of Natural Sands Related to Their Physical Attributes," by C. F. Zietsman, which appeared in the May 1957 JOURNAL.

p. 1052—Square brackets on both right and left hand side of Eq. (10) indicate absolute quantities, not algebraic differences; i.e., they are used in a positive sense in solving the equation, regardless of whether the quantity within the brackets is positive or negative.

The equation may better be written as follows:

$$\left|\frac{W_4}{10} - 37.2\right| + 9 = \sqrt{9^2 + 13.2^2 - \left(13.2 - \left|100J - 62\right|\right)^2} \quad \ldots\ldots (10)$$

* * *

The following correction should be made in "Considerations in the Selection of Slab Dimensions," reported by Subcommittee II, ACI Committee 325, in the November 1956 JOURNAL.

p. 446—The expression

$$\frac{\frac{E_c h^3}{k (a_e + b_e)^4}}{2}$$

in Eq. (4) should read

$$\frac{E_c h^3}{k\left(\frac{a_e + b_e}{2}\right)^4}$$

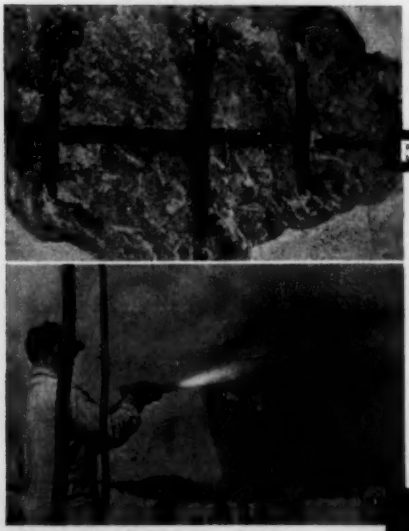
The following correction should be made in "Proposed Recommended Practice for Design of Concrete Pavements," reported by ACI Committee 325, in the February 1957 JOURNAL.

p. 726—The expression

$$\frac{\frac{E_c h^3}{k (a_e + b_e)^4}}{2}$$

in Eq. (4) should read

$$\frac{E_c h^3}{k\left(\frac{a_e + b_e}{2}\right)^4}$$



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NATIONWIDE SERVICE

Precision boring instrument

Newly developed portable precision boring instrument Kor-It, Jr. with motor and attachments has total weight of 36 lb. It uses diamond bits and cut holes at speeds up to 6 in. per min. Compactness permits its use in areas previously considered inaccessible. It drills at any angle and up to depths of 8 in., and to greater depth with extensions. Cutting is done to exact diameter desired and no chipping is required, the manufacturer states.—J. F. Hamlin Co., Inc., 746 Ellis St., San Francisco 9, Calif.

Dust control systems

Bulletin tells how cement plants can suppress dust at source before it arises in atmosphere emphasizes advantages of "chem-jet" system. Multiple jets apply small amounts of surface active agent so that dust is retained on material being handled. Other data are presented on hydro-precipitator scrubber designed for removal of dust particles ranging below 5 microns in particle size.—Johnson-March, 1724 Chestnut St., Philadelphia 3, Pa.

Design manual

Design manual has been developed for jobs using perlite insulating concrete and/or perlite-gypsum plaster. The 80-page loose-leaf volume contains related specifications, data, and detail drawings for complete roof and floor systems, curtain wall back-up, and other building sections.—Perlite Institute, 45 W. 45th St., New York 36, N. Y.

Hydra-hammer

Larger engine and heavier hammer on model 8PHH-1000 Ottawa "Hydra-Hammer" effect greater production on all pavement breaking, cutting, and backfill tamping jobs, says manufacturer. New model includes such features as traverse tower arrangement, hydraulically controlled laydown, engine cover, rear fenders, adjustable seat, flange type tools, and combined power brake, and clutch control. The 7500 ft-lb blow developed by model is actually controlled so maximum blow, or minimum blow as little as 100 ft-lb can be used at operator's will — Ottawa Steel Division, L. A. Young Spring and Wire Corp., Ottawa, Kan.

Vapor barrier coating

A coating, designed to control transmission rate of water vapor through any type of building material, is guaranteed by the manufacturer to retard rate of vapor transmission to not more than 0.3 grain per sq. ft per hr at 1 in. mercury vapor differential (approximately 160 grains) at 80 F. A vinyl resin solution, the coating can be applied to interior or exterior surfaces. Said to be easily applied to any type of building material, it is available in satin aluminum and colors.—Ellis and Watts, Inc., P.O. Box 33, Cincinnati 36, Ohio.

Testing equipment catalog

Available from manufacturer is 48-page catalog No. 21 that describes balances and weights, cement and concrete testing equipment, drying ovens, soil testing equipment, testing sieves and shakers, and miscellaneousequipment.—Humboldt Manufacturing Co., 2104 N. Whipple St., Chicago 47, Ill.

Trailer used as batch carrier

Stabilift "frameless" semi-dump trailer can be used as a batch carrier on concrete paving jobs. Field performance reveals that the 20-ton capacity permits the

trailer to haul seven 5500-lb batches per trip. Danger of tipping has been eliminated and unit can be used on all type terrain according to manufacturer. Operator can quickly move this trailer in and out of skip and discharge each batch while paver operates at full capacity, it is claimed.—Cook Bros. Equipment Co., 3334 San Fernando Road, Los Angeles, Calif

Masonry panel

A lightweight masonry panel, called "Tecfab," is made from corrugated steel core embedded in precast perlite concrete. It weighs 16 to 17 lb per sq ft and is 4 in. thick. Its use is suggested for complete wall, interior partition, floor, and roof systems. Panel usually features exposed aggregate on its exterior face; interior face, of smooth white perlite concrete, has natural finish which can be painted.—Tecfab, Inc., Beltsville, Md.

Cement base paint

A cement base paint called "Waterfoil KB-55," designed to prevent water seepage in basements, silos, and elevator pits, has been introduced. Catalogs describing this and other products for industrial maintenance and new construction are also available.—A. C. Horn, Inc., 104—44th Ave., Long Island 1, N. Y.

ALPHABETICAL LIST OF ADVERTISERS

(Page Numbers refer to News Letter)

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Lone Star Cement Corporation ..... Inside Front Cover

Marathon Corporation 23

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Sika Chemical Corporation 8

Soiltest, Incorporated 12

Techkote Company, Incorporated 18

Western Waterproofing Co., Inc. 24

The Institute assumes no responsibility for the claims of advertisers. The advertiser is made responsible in the belief that his place in the field will be determined by the public's ultimate measure of his exercise of that responsibility.

Tetrapod forms

Steel forms for mass producing cast tetrapods for breakwater construction are now being fabricated. Tetrapod is an integrally cast, four-armed block.

Each of the identical arms has the shape of a truncated cone and is attached to a central core. Developed and first used by the French, tetrapods have been used most successfully in breakwater construction where heavy seas are encountered and a large volume of natural construction materials are readily available.

Fundamental advantages are that less volume of rock and concrete is required. Other advantages cited by the firm are that work can be carried out year-around in most areas, casting during inclement weather, and placing the pods when weather permits. Forms for casting 31-ton tetrapods, largest yet to be made in the United States, are now being manufactured. —Blaw-Knox Co., 300 Sixth Ave., Pittsburgh 22, Pa.

Waterstop test report

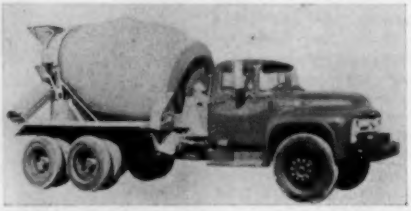
Technical Report No. 1 issued by Electrovert, Inc., summarizes tests made to determine how design of cross section of a waterstop affects bonding of waterstop to concrete. This report discusses "holding-power" of various designs of waterstops.—W. R. Meadows, Inc., 7 Kimball St., Elgin Ill., or Tecon Products, Inc., 304 Alaskan Way, Seattle, Wash.

Portable diamond concrete driller

Equipped with a water swivel, high-speed portable diamond drilling machine is said to drill 2 in. holes at up to 3 in. per min. Holes as small as 3/16 in. can be drilled in hardest of building materials says manufacturer. Flexible cable drive allows freedom for drilling holes in any position and eliminates necessity of operator holding a heavy unit. Machine operates on 110 volts.—Powder Tool Fasteners, Inc., 5349 W. Madison St., Chicago, Ill.

Four-speed truck mixer

New 5- to 7-cu yd Mixomatic truck mixer features front engine power take-off with automatic transmission which provides four discharge speeds. Two levers,



throttle and transmission, simplify the controls. Additional design and equipment features are the new hopper for faster charging, a three-way quick shut-off water value and splash guards from fender to frame for drum protection. Total weight is approximately 6800 lb.—Willard Concrete Machinery Co., 11700 Wright Road, Lynwood, Calif.

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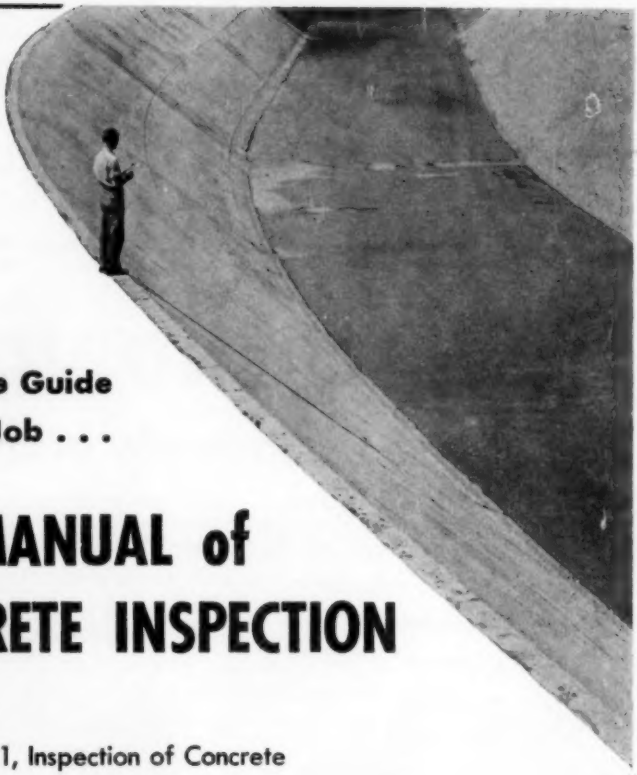

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